ABSTRACT

Title of Thesis: PERFORMANCE OF REPAIRED REINFORCED CONCRETE COLUMNS UNDER AXIAL LOADING

Murat "Rod" Seyidoglu, Master of Science, 2020

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The purpose of this research was to determine the performance and axial load capacity of repaired reinforced concrete columns under compressive loading. While various concrete column strengthening methods, using concrete or carbon fiber reinforcing polymer (CFRP) jacketing, have been exhaustively researched, there is an insufficient amount of research regarding the performance of column repairs utilizing concrete removal and replacement. To research this topic, nine reinforced, one-third scale concrete columns were cast; and, six of these columns were repaired with conventional concrete repair methods recommended by the American Concrete Institute's Guide to Concrete Repair (546-14). Shallow (extending up to the exterior faces of column ties) and deep repairs (extending behind the vertical bars) were performed at the bases of the columns by chipping the concrete with a handheld chipping hammer and patching with a shrinkage compensated repair mortar as it is commonly done for repairing corrosion related spalls. Results of the axial compression loading tests and failure patterns of the repaired columns were compared to that of the control samples. It was found that all repaired concrete columns achieved comparable load capacity values and exhibited the same failure mode as the intact columns.

PERFORMANCE OF REPAIRED CONCRETE COLUMNS UNDER AXIAL LOADING

by

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Dedication

To Chrissy

Acknowledgments

I would like to thank Dr. Zhang for his continuing support of this research, Bill Lyons and Euclid Chemicals for providing me with the repair materials and my coworkers for tolerating my academic schedule over the years. I would like to particularly thank Tina Berger for allowing me to use her facilities for rebar fabrication.

I hope that this research will make a meaningful contribution to the relatively new concrete repair industry and become a valuable resource on the topic. By the collaborative and selfless efforts of the structural engineers, structural repair industry has gained its foundation; now it is our duty to improve it and fill in the gaps.

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Nomenclature

A_c – Area of Concrete (in^2)

A_g – Gross Sectional Area (in^2)

A_{st} or A_s – Area of Steel (in^2)

d – Diameter (in)

E – Modulus of Elasticity (kips/in^2 or lbs/in^2)

E_c – Modulus of Elasticity of Concrete (lbs/in^2)

f'_c – Average 28-day Compressive Strength (lbs/in^2)

f_y – Yield Strength of Steel (kips/in^2)

L – Length (in)

I – Moment of Inertia (in^4)

P – Applied Load (kips or lbs)

P_n – Nominal Column Capacity (kips)

P_u – Ultimate Column Load (kips)

S – Sectional Modulus (in^3)

v – Shear Flow (kips/in)

w_c – Concrete Density (pcf)

w/c – Water to Cement Ratio (%)

ε – Strain (in/in)

τ – Stress (kips/in^2 or lbs/in^2)

τ_{creep} – Creep Stress (kips/in^2 or lbs/in^2)

Δ – Axial Deformation (in)

Chapter 1: Introduction and Background Information

1.1 Research Purpose and Available Literature

The purpose of this research was to investigate the axial load capacity and behavior of concrete columns repaired with chipping and patching methods prescribed by various institutions such as the British Concrete Society and the American Concrete Institute. Such methods outlined by these institutions are a derivation/summary of commonly utilized methods by the engineers specializing in the repair of concrete throughout the world (mainly developed based on principles of structural engineering), and there are many examples of successful concrete repairs performed with such techniques. A number of these professional organizations throughout the world have various dedicated committees and publications regarding repair of concrete structures along with provisions for repairing axial load carrying members; however, none of which contains results derived from actual load tests. In addition to concerns with repairing the concrete itself, scattered information related to stiffness, shrinkage, and deformation of the repaired members are available in the published literature. Particularly ACI Committees 364 and 546's Publications (*ACI 364.5T-10 Importance of Modulus of Elasticity in Surface Repair Materials, ACI 546R-14 Guide to Concrete Repair*) provide some relevant information related to load sharing between new and existing materials based on their stiffness as well as modulus of elasticity. Both documents indicate that materials with higher stiffness and elasticity would sustain more load **(Figure 1-3)** and sustain deformation; they provide further recommendations for the selection of repair materials and repair techniques for various types of damage.

However, there has not been any actual load testing done on columns to find out the axial load capacity of repaired columns.

While there are many guides and significant research related to different concrete and FRP jacketing schemes, there is minimal research into repairing concrete columns without jacketing. Conventional chipping and patching methods are frequently used in isolated structural column repair applications, for removal and replacement of spalled concrete associated with corrosion of reinforcement. Unfortunately, these basic methods are not well researched, and there is very limited research done to investigate the behavior of columns which have undergone such repairs.

This research was conducted to investigate a topic commonly encountered in the professional practice and written to clearly explain the findings along with recommendations without congesting the material with redundant information.

1.2 Testing Summary

In order to investigate the capacity of the repaired columns, a total of (9) normal weight, 8-inch diameter and 36" long columns were cast (**Figure 1-1**) by utilizing a premixed, and commercially available, 3,500 psi concrete mixture with pearock aggregate (1/2" max. size); details of the samples are as follows:

- 3 columns as reference with no repairs (Control).
- 3 columns repaired up to the stirrups cages at by removal and replacement of concrete at 6"x12" areas at the bases of the columns (Referred to as shallow repairs – **Figure 1-2**).

- 3 columns repaired 1-inch beyond the column cages at 6”x12” areas at the bases of the columns (Referred to as deep repairs – **Figure 1-2**).

All columns were left to cure for 28 days and attain a compressive strength of 3,500 psi before the performance of repairs. A relatively low compressive strength was selected to replicate a concrete column that would be susceptible to corrosion due to low density (and compressive strength). Six of the cast samples were repaired with a non-shrink, self-consolidating repair mortar to ensure that the load transfer between the new and existing sections of the concrete was not adversely affected by shrinkage and load distribution to the whole column would take place after the axial load application. Demolition was performed by utilizing a lightweight electric chipping hammer to remove the selected sections of the columns without causing cracks or fissures. The compressive strength of the concrete samples was monitored by utilizing a calibrated Swiss hammer periodically. At the time of the load testing, control columns were allowed to cure for 56 days to obtain their expected compressive strength and complete their drying shrinkage cycle.

Testing was performed by utilizing 1/3 scale columns to replicate 24-inch diameter columns, which can be commonly found at an open multi-story parking garage structure undergoing structural repairs due to corrosion / spalling. Confinement of the columns was accomplished by placement #3 ties at maximum 8-inches on center, with the exception of the ends of the columns where the ties were placed at approximately 3” o.c. max (**Figure 1-1**). Zero load eccentricity was used to gain a better understanding of the load sharing between old and new concrete and avoid generating different stresses at different sections of the columns related to eccentricity. Approximately 3-inches of concrete cover was provided at one end of the vertical bars to replicate a pier condition, or extreme exposure, which can also be found at an exposed column exposed to harsh environmental effects or placed against earth. ASTM A615, Grade 60 rebar was utilized at all columns. Six No.4 bars (As=1.2 in^2 or 4.2% reinforcement ratio) were placed to keep the column reinforcing within the 0.01Ag to 0.08Ag in accordance with ACI 318 (*2014*) Section 10.6.1. Further discussion is provided regarding the contribution of these bars in the preceding sections.

1.3 Analytical Summary and Detailing

A review of the ACI 318 *(2014*) for the design of concrete columns, expected capacity, and how compliance with such requirements is ensured, is necessary to elaborate the chosen rebar detailing for the tested columns. <u>It has to be noted that the purpose of this experimental testing was not to verify the conformance of the columns with the code (or make a comparison against the expected capacities); rather, testing was done primarily to find out whether or not there is proper load transfer between the repaired and unrepaired sections of the columns and observe their failure pattern.</u>

Compressive strength of concrete columns is determined in accordance with ACI 318 (*2014*) Section 10 by considering the effects of the materials, boundary conditions as well as lateral bracing (slenderness effects). In order to provide sufficient confinement to columns, Section 25.7.2 of ACI 318-14 requires stirrups to be placed at a spacing to comply with the following:

- Clear spacing to be no less than (4/3) x diameter of the aggregate.

- Center to center spacing not to exceed 16 x diameter of the horizontal rebar or 48 times the diameter of the tie bar or the least dimension of the column.

Furthermore, Section 25.7.2.2 requires the use of #3 stirrups for enclosing #10 or smaller bars.

Section 25.7.2.4.1 provides the requirements for the circular tie reinforcement to ensure that the vertical splitting and loss of tie restraint are prevented under compressive loading. Adjacent circular ties are prohibited from engaging the same longitudinal bar to prevent the formation of a vertical crack, which can spread vertically if the ties are not staggered. Also, the ends of the ties are required to be lapped at a minimum of 6-inches to reduce the slip of the stirrups and hooked around vertical reinforcement with standard hooks.

To comply with the detailing requirements, #3 rebar ties with alternating hooks were placed at max. 8” o.c. at full height of the columns (except for the ends where the first tie was placed 3” from the ends), as shown in **Figure 1-1** below and **Figure 3-1**.

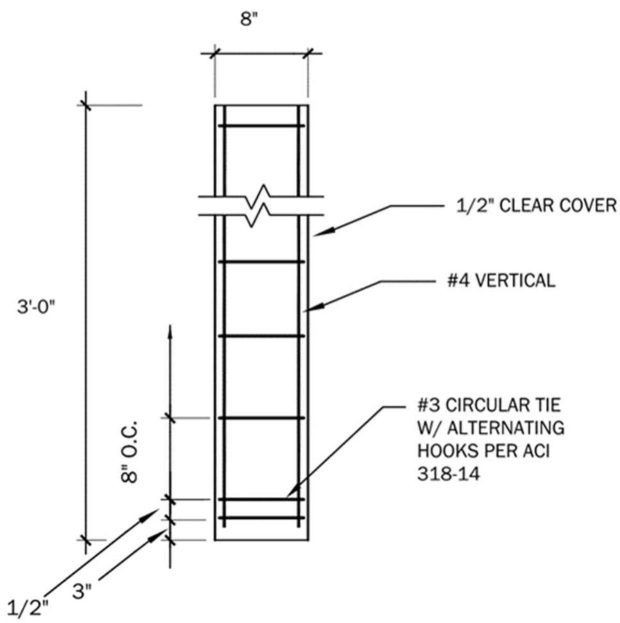


Figure 1-1-Vertical Section of Tested Columns

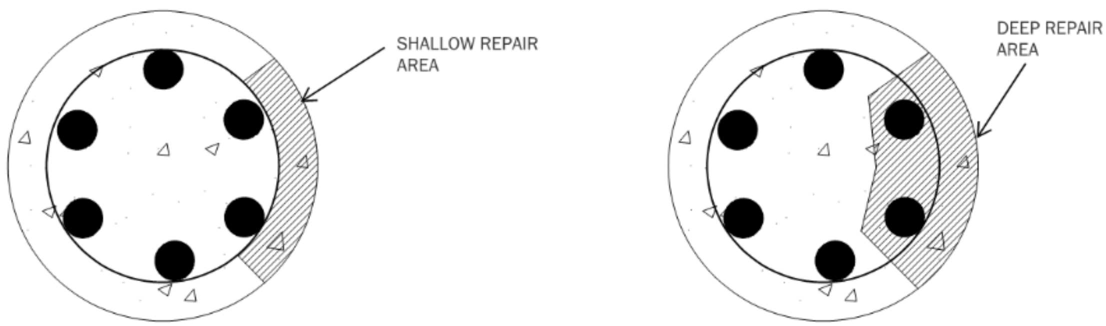


Figure 1-2-Shallow and Deep Repair Areas

1.4 Theoretical Axial Capacity of Reinforced Concrete Columns per ACI 318

According to Section 22.4.1 of ACI 318 (*2014*), the axial compressive strength of concrete columns is determined with the following formula:

$$\phi P_n = \phi 0.85 f'_c \text{ x } (A_g - A_{st}) + f_y \text{ x } A_{st}$$

This formula establishes the available capacity based on the properties of each material. It requires the use of an 0.85 factor to accommodate accidental eccentricities in columns at eccentricity to depth ratios of 5 to 10 percent. While the reinforcing steel generally has a smaller contribution than the concrete, at higher reinforcement ratios, this contribution becomes significant. By utilizing this formula, tested concrete columns would have a nominal compressive strength of:

$P_n = 0.85 \times 3{,}500 \text{ psi} \times (50.24 \text{ in}^2 - 1.2 \text{ in}^2) + 60{,}000 \text{ psi} \times 1.2 \text{ in}^2$

$P_n = 217.89$ kips

This formula assumes that the vertical steel was allowed to provide full compressive resistance, and the steel would be properly developed to gain its yield strength; however, in the case of 1/3 scale piers cast not cast into a column foundation, development of the steel's yield strength is not possible. Under these conditions, i.e., the steel development cannot be attained, the appropriate compression capacity of the columns can be calculated as:

$Pn = 0.85 \times 3{,}500 \text{ psi} \times (50.24 \text{ in}^2) = 149.464$ kips and

$Pu = 3{,}500 \text{ psi} \times 50.24 \text{ in}^2 = 175.84$ kips (pure compression without eccentricity)

Disregarding the contribution of the rebar to the axial load capacity of concrete columns cast without a foundation (or dowels going into foundations as commonly done in precast columns) is more appropriate than utilizing such capacity as there is no load transfer mechanism from the base of the column to the foundation beyond bearing. As noted above, vertical rebar will not be developed at the base of the column without the bars being extended into a foundation.

The requirement for a positive connection between foundations and cast-in-place columns is clearly explained in ACI 318 (*2014*) Section 16; according to Section 16.3.1.1, factored forces at the bases of columns need to be transferred with bearing strength and concrete; furthermore, Section 16.3.4.1 requires a minimum As of $0.005A_g$ which equates to 0.25 in^2 (could have been accomplished by placement 3-#3 rebar in the tested columns) for cast-in-place columns to accomplish a load transfer mechanism.

ACI 318 (*2014*) provisions for precast concrete columns should also be considered in this case as the columns were cast in controlled conditions and tested without a foundation. Again, the use of the rebar strength would not be appropriate as the rebar was not connected to develop the full design strength at the base of the columns. Particularly ACI 318 (*2014*) Section R16.3.3.4 reads: "*In the common case of a column bearing on a footing, where the area of the footing is larger than the area of the column, the bearing strength should be base of the column and the top of the footing. In the absence of dowels or column reinforcement that continue into the foundation, the strength of the lower part of the column should be checked using the strength of the concrete alone.*"

1.5 Modulus of Elasticity and Stiffness of Columns

Section 19.2.2.1 of the ACI 318 (*2014*) permits the calculation of the modulus of elasticity as:

$E_c = w_c^{1.5} \times 33 \times (f'c)^{0.5}$, which provides a modulus of elasticity of 3.41×10^6 psi with a concrete density of 145 pcf and 3,500 psi compressive strength.

While modulus of elasticity is a fundamental property in compressible columns and determination on slenderness effects, it has to be considered in conjunction with the sectional properties of the repair material in column repair applications. Load sharing between two different materials under compressive loading will depend on their sectional modulus and stiffness (EI) as shown in **Figure 3** below. From this figure, it can be concluded that the theoretical compressive strength of the repaired column will be equal to the compressive strength of the unrepaired columns if the stiffness of the remaining section is matched. By considering the total section modulus of the column section as:

$EI_{total} = (E_{remaining} \times I_{remaining} + E_{repaired} \times I_{repaired})$

Load at the remaining section of the column = $P_{applied} \times (E_{remaining} \times I_{remaining}) / EI_{total}$

Load at the repaired section of the column = $P_{applied} \times (E_{repaired} \times I_{repaired})$

where

$E_{repaired}$ is the sectional modulus of the repaired section of the column.

$I_{repaired}$ is the moment of inertia of the repaired section of the column.

$I_{remaining}$ is the moment of inertia of the remaining section of the column.

$E_{remaining}$ is the modulus of elasticity of the remaining section of the column.

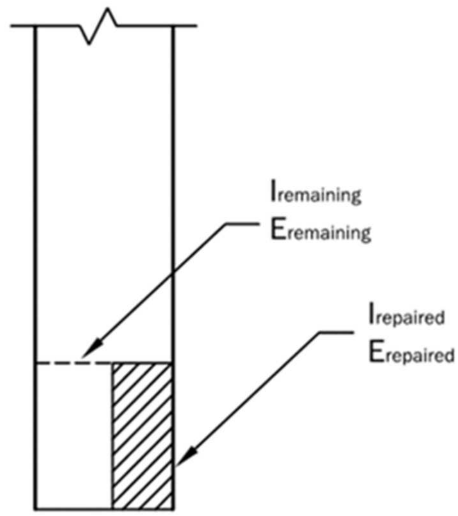


Figure 1-3- Section Moduli and Stiffness for Repaired and Remaining Sections

Modulus elasticity is a consideration in material selection due to potential issues with the deformation of different sections of concrete. Considering that $\Delta = PL/AE$, deformation (and axial capacity) of different sections of the column will depend on each section's modulus of elasticity. In large repairs with significantly different materials, this deformation may be significant enough to create additional bending stresses within the column if the resultant of the force moves away from the column centroid at a considerable amount (**Figure 1-4**). But, it is unlikely that this additional moment would be an issue, as explained below.

The Author of this thesis recognizes that the effects of modulus of elasticity are not significant in isolated shallow column repairs. If the amount of repair material is limited, and the material is surrounded by existing concrete, load transfer through the section is the primary concern. Concrete itself does not behave completely uniformly, sectional differences exist within different sections of concrete columns, and the use of repair materials with similar elastic moduli should not alter that behavior. If a repair material is chosen with a significantly different elastic modulus, an internal moment may be created by the attraction of load to stiffer sections of the columns would, unlikely, be an issue due to the fact that axial and flexural strength of the overall column will be higher due to use of higher strength concrete in the repaired section.

It needs to be noted that concrete columns commonly fail due to the effects of overloading, slenderness, insufficient confinement, or spalling of the concrete from the rebar. To the Author's knowledge, there have not been any recorded instances where the use of concrete with different modulus of elasticities led to failures.

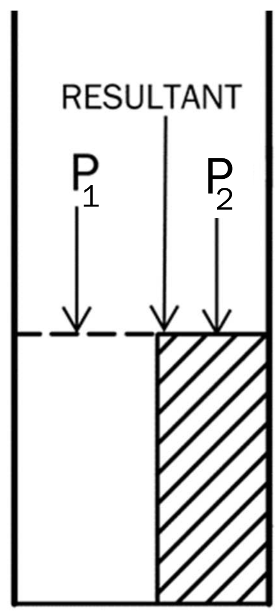


Figure 1-4- Location of Resultant Force in Column Section

1.6 Shrinkage of Concrete

Shrinkage of concrete is a crucial consideration literally in every structural repair project and for every type of member being repaired. It is necessary to elaborate on the types of concrete shrinkage before the material selection criteria (shrinkage compensating, self-consolidating, and inclusion of fibers) can be appropriately explained, and a detailed discussion on the testing procedures can be made. While the purpose of this research is not an exhaustive study of the effects of shrinkage in concrete repairs, it is very significant. During the preparation of the samples, shrinkage was controlled by proper placement, curing, and material selection. Three primary types of shrinkage cracking associated with: i) plastic shrinkage, ii) drying shrinkage (also creep) and ii) thermal expansion/contraction, significantly affect the durability of concrete and will be elaborated as necessary background information:

1.6.1 Plastic Shrinkage

Plastic shrinkage is related to rapid evaporation of moisture from the surface of the concrete soon after it is poured and the loss of the moisture through the substrate over which the concrete is placed (i.e., concrete placed over a vapor barrier would be less likely to sustain this type of cracking). As the name suggests, such cracking occurs while the concrete is still in a plastic stage and before it hardens.

After the concrete is poured, mixed in water starts evaporating from the top and gets replaced by the bleed water within the mix. Since water is less dense than the remainder of the materials making up the concrete, water will seep up as the rest of the materials settle down. The water which reaches to the top surface is referred to as “bleed water.” More bleed water is observed with mixes high w/c ratio and fines as more water will float to the top as the finer particles settle. Also, the evaporation rate is a decisive factor affecting the plastic shrinkage occurring soon after the pour. While a myriad of factors affect the rate of evaporation, the surface area of the concrete, ambient temperature, wind speed, temperature of the mix, and humidity are the main contributors. The evaporation process initiates cracking by causing drying shrinkage at the top surface of the concrete. After the bleed water at the surface evaporates quickly, it causes surface tension (due to thermal expansion and contraction of the surface), which results in cracking. As the evaporated bleed water gets replaced from water within the mix, this drying and shrinkage cycle continues until the hydration process is completed. Accordingly, high levels of evaporation are observed under high temperatures and continuous high winds during concrete pours.

Plastic shrinkage cracks can be identified by their unique appearance related to their formation. As explained above, this process occurs when the concrete is still plastic; the rupture of the semi-hardened cement matrix causes smooth edges along the cracks. Such cracks remain shallow as the surface tension is generally not sufficient to create significant stresses in the deeper sections of the concrete; also, the crack penetration is stopped by the presence of the aggregates limiting the extension through the matrix. While plastic shrinkage cracks are commonly observed in structural slabs and are a more significant concern in such applications, they can cause issues in column/beam repair applications where the surfaces of the repaired areas are exposed to elements during curing. While gravity-fed epoxy applications are efficient in filling such cracks, a low-pressure epoxy injection may be necessary for vertical or overhead cracks (at the soffits of slabs). In many cases, such cracks do not pose a structural problem and are simply sealed to prevent corrosion.

Figure 1-5 Plastic Shrinkage Cracking at a Slab on Grade

1.6.2 Drying Shrinkage

Drying shrinkage occurs after the concrete hardens and is associated with the loss of moisture within the concrete matrix. Unless the concrete remains completely saturated and unrestrained, loss of moisture from the hydrated cement paste and internal stresses caused by such volumetric changes will cause drying shrinkage cracks. At reinforced concrete structures, these changes are ordinarily unavoidable as restraint from the structure itself cannot be removed without compromising structural integrity. The complex relationships within the components of the concrete causing such cracking will not be elaborated in detail as such a discussion is beyond the scope of this research. However, it is vital to elaborate on different factors causing drying shrinkage and how they can be reduced (and were reduced during the preparation of the column samples) and the types of concrete vulnerable (which are not suitable in repair applications). Mixes with high water-cement ratios, low cement content, and small aggregates commonly sustain more drying shrinkage cracking. Utilization of low w/c ratios, larger aggregates, fiber reinforcement, distributed reinforcement will reduce the propagation of cracks through the concrete section and internal stresses associated with the loss of moisture (large aggregates reduce the total amount of water in the mix in addition to stopping the crack propagation).

Drying shrinkage cracks often present themselves in a random or s0piderweb shaped pattern in slabs and vertical wall surfaces. ACI 224.1R (*2012*) provides information related to the assessment, evaluation, and repair of such cracks by various methods.

Figure 1-6-Drying Shrinkage Cracks at a Precast Double-Tee

1.7 Creep

Creep and drying shrinkage are both longer-term issues associated with volumetric changes after the pour. In the Author's opinion, one issue cannot be explained without elaborating on the other. In the most simplistic terms, creep is the deformation of the concrete under sustained loading. However, as ACI 209R (*1992*) elaborates, this is a more complicated phenomenon involving deformation effects due to the material and the reduction of moisture at the same time requiring adjustments for different material properties and curing conditions. While the effects of creep and not relevant in repairs of slabs, beams, and other flexural members, it becomes somewhat of consideration in column repairs due to the repair materials sustaining direct compression for prolonged periods. Thus, ASTM C512 (*2010*) testing is commonly performed on repair materials to determine their creep behavior used in compressive members. This test is performed on 6-in diameter cylindrical samples cured in moist conditions (where free water is present at the surfaces of the samples at all times) after 7 and 28 days. Commonly used high-performance materials in column repairs generally exhibit average creep strains in the vicinity of 0.03 µstrain/psi at 28-days.

For a sustained stress of 3,500 psi, the expected creep strain would equate to $0.03 \times 10^{-6} \times 3{,}500 = 0.000105$. This number itself does not look large; however, it could cause issues in longer (in dimension) repair applications. For instance, if the repair area is 10-ft long, this creep would equate to a 0.013-in change in length and tensile forces associated with this dimensional change, $F_{creep}=\varepsilon x A$. This force will act along the bond line between the new and old concrete and may result in cracks as well as delamination. Though not very common, the Author has experienced creep failures in repair applications with insufficient bond between new and existing concretes.

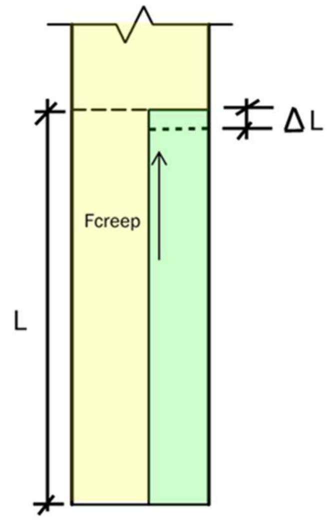


Figure 1-7-Creep Stresses at Bond Line

While ASTM C512 (*2010*) is a useful resource, its applicability in material selection for column repairs is minimal; its testing methodology is limited to a material's compressive stresses being limited to 40 percent of its capacity and the material being cured in water. In almost all real-world applications, these two factors do not take place. The deadweight of the structure generally causes compressive stresses exceeding 40% of the f'_c (particularly at the lowest levels of the structures where most of the concrete deterioration is commonly observed), and complete moist cure of repairs is impractical in the field. The presence of vertical column reinforcement is another factor that is not considered in the ASTM testing. Reinforcement within the repair area will help to resist the internal creep stresses within the patch area through shear friction between the rebar and concrete. With its many shortcomings, this document gives a basis to understand the creep characteristics of the repair material. It could be a useful resource in deep applications with limited or no rebar.

1.8 Thermal Expansion / Contraction

The effects of thermal expansion and contraction are well known and studied. Concrete with a thermal expansion coefficient of approximately 7 to 8 x 10-6 in/in/°F will sustain tensile stresses under temperature effects. ACI 318 (*2014*) provides minimum reinforcement limits for shrinkage and temperature effects for various types of members. For 1-way slabs, it requires a minimum transverse reinforcement ratio of 0.0020 with Gr. 60 reinforcement. For reinforcement with yield stresses exceeding 60 ksi, the greater of $0.0018x60{,}000/f_y$ or 0.0014 is specified. For walls, minimum transverse reinforcement limits, similar to that of 1-way slabs, are provided in Table 11.6.1. In addition, skin reinforcement for deep beams (Section 9.9.3.1) and detailing practices to distribute the reinforcement for minimizing the effects of temperature are specified for all types of members. While the intent of ACI 318-14's design requirements appears to be the provision of reinforcement perpendicular to potential cracks to reduce their with (or eliminate the crack initiation under low superimposed load levels), crack propagation through thermal effects can only be eliminated under hypothetical unrestrained conditions which are do not represent normal structures. Expansion/control joints are commonly provided to allow the structures to move and reduce cracking if deemed necessary. Various concrete design codes suggest different spacing limits for such joints, which ultimately need to be determined by the engineers in charge of the design/analysis.

In the repair of compression elements, thermal expansion/contraction has a limited effect. As both the repaired and unrepaired concrete will sustain significant compressive loading and go through the same thermal expansion/contraction cycles, this property will not be significant. However, thermal expansion/contraction is an essential consideration in the repair of large areas in conjunction with joint spacing.

1.9 Bond Strength between New and Old Concrete (Shear Friction)

Shear friction is the main mode of load transfer in concrete repair applications. Shear flow (v) within any cross-section needs to be considered and resisted for the concrete to resist the superimposed loads. From basic mechanics, it is known that:

$v = VQ/I$

Where v is the shear flow (kips/in)

V is the superimposed shear (kips)

Q is the first moment of area (in^3)

I is the moment of inertia of the whole section (in^4)

Within a concrete patch, this force is resisted and can be calculated per ACI 318 (*2014*) as:

The least of ($0.2f'_cA_c$) or $(480+0.08f'_c)A_c$ or $1600A_c$ for normal weight concrete cast against concrete intentionally roughened to ¼" amplitude.

and

$0.2f'_cA_c$ or $800A_c$ for all other cases.

ACI 318 (*2014*) conservatively states that shear transfer needs to be resisted by shear friction reinforcement (A_{vf}) provided perpendicular to the failure surface by ignoring the shear friction at the concrete to concrete surfaces. This requires all shear transfer to be made by the shear reinforcement exclusively. While this reinforcement will likely be present beams (in the form of stirrups), columns and beams will not contain such reinforcement, and compliance with this provision is not likely. Earlier revisions of ACI 318 less conservatively refers to the research done by Mattock, Li, and Want (Mattock et al. *1972 and 1976*) and more accurately provides shear resistance to be calculated as:

$$Vn = 0.8A_{vf}f_y + A_cK1$$

Where K1 was found out to be 400 psi for normal-weight concrete, 200 psi for all-lightweight concrete ad 250 psi for sand-lightweight concrete. It also uses the 0.8 factor for the coefficient of friction. When the existing concrete surfaces are appropriately prepared and roughened before new concrete placement, the use of this 400-psi shear capacity is more appropriate than completely ignoring the shear friction through concrete-to-concrete friction. This mode of failure particularly makes sense in repairs sustaining vertical loads where the patches are restrained, and the load transfer is enabled by the shear friction as well as bearing of the repair material within the repaired section.

While the horizontal shear topic requires a significant amount of research, ACI 562 (*2013*) includes some guidance on this matter without significant testing or research performed on scaled size structural members. It indicates if the v_u is below 60 psi, no shear friction reinforcement is necessary, and various load tests should be made to determine the shear capacity of the bod by equating to the tensile capacity in a conservative approach. This recommendation appears to be very conservative when compared to the research done by (*Mattock, et. al 1972 and 1976*).

1.10 Durability Considerations

The durability of concrete against various external effects is an essential consideration in repair applications requiring an assessment of the external conditions as well as the customization of the concrete mixes, air content, clear cover, and if needed, provision of protective coatings. While denser concrete mixtures with a higher water to cement ratios protect embedded reinforcement by resisting penetration of fluids and ions into concrete, exposure to various elements such as chlorides, sulfates, nitrates, and acids require concrete mixtures to be densified by the inclusion of silica fumes, fly ash or slag along with increasing the clear cover.

ACI 318 (*2014*) provides five exposure categories (F, T, S, W, and C) based on most commonly encountered situations (Freeze and Thaw, Sulfates, Water -without chlorides or freeze/ thaw- and Corrosive) and classifies each of these categories from 0 to 3 based on the intensity of the exposure. Furthermore, ACI 318 (*2014*) Table 19.3.2.1 provides the minimum requirements for compressive strength, w/c ratio, air content, and the limits on cementitious materials as well as the concrete cover. ACI 201.2R (*2008*) provides a more detailed guideline for considering alkali-silica reactions, various types of chemical attack, and abrasion, which supplements ACI 318 (*2014*).

A lesser considered but as important issue carbonation of concrete. Carbonation occurs due to penetration of CO_2 into concrete and formation of carbonic acid, which causes

corrosion of embedded reinforcement. According to the National Precast Concrete Association, 0.039 inches of carbonation per year is expected (*Hanson, 2015*). Carbonation beyond this rate is considered to be a sign of low density, which would indicate concrete being susceptible to chemical attack and chloride intrusion. Also, the depth of carbonation penetration can help to determine the age of structures when if a correlation between the density and carbonation can be made for the concrete being investigated.

Figure 1-8-Depth of Carbonation at a Beam

Chapter 2: Concrete Repair Methodology

2.1 Assessment of Deteriorated Concrete

The first step in determining appropriate repair for concrete deterioration is the performance of an assessment to figure out what caused the damage and the extent of the damage to be repaired. Accompanied by visual observations, destructive and non-destructive testing methods are utilized to assess concrete structures. While an introduction will be made to such concrete repair techniques, the focus of this research is the and axial capacity of repaired reinforced columns; thus, the information was tailored to provide the appropriate repair methodology for concrete columns to accompany the necessary material information given in the preceding chapter.

2.1.1 Typical Issues Observed in Concrete Columns

In most building structures, column damage is observed at lower levels (such as basements and entrances) or sections of the structure where the columns can absorb chloride laden water brought in by the elements or, in the case of parking garages, vehicles. Water, deicing salts, and other contaminants penetrate the concrete, which causes the steel reinforcement to corrode (and surrounding concrete to spall). Furthermore, chloride ions can already be present in the concrete due to the use of admixtures that contained calcium chloride when the building was constructed. After the delamination, reinforcing steel becomes more exposed to water and salts, leading to an accelerated rate of deterioration. As the reinforcing steel corrodes, it loses cross-section and further de-bonds from the concrete.

Corrosion of the rebar is an electrochemical process which occur at the active portions of the embedded reinforcement, in presence of moisture and oxygen. This electrochemical corrosion process is explained as:

$2Fe ==> 2Fe^{2+} + 4e^{-}$

$2H_20 + O_2 + 4e^{-} ==> 4OH^{-}$

$2Fe^{2+} + 4OH^{-} ==> 2Fe(OH)$

Where the byproduct Fe(OH) is rust occurring at the surface of the rebar. Fe(OH) displaces the surrounding concrete and causes cracks.

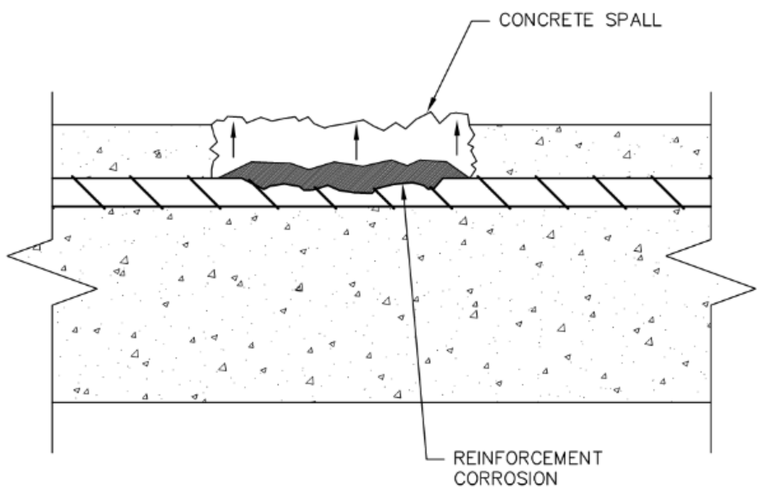


Figure 2-1-Concrete Spall Initiation

As can be seen in the chemical equations above, three key components required to initiate corrosion are iron, oxygen, and water. Usually, chloride ions are mistakenly attributed to being directly consumed in the electrochemical process producing rust; however, they are only contributors to this process. Chloride ions accelerate corrosion by reducing the passive protective layer around the rebar, which is believed to exist. When the alkalinity of concrete decreases in the presence of chloride ions, this protective layer is believed to deteriorate, which leaves the rebar susceptible to corrosion. All of the issues elaborated in Chapter 1, associated with the cracks and deterioration of concrete, are vital parts of this electrochemical corrosion process (and other deterioration mechanisms associated with carbonation and chemical attack). When there is insufficient cover or cracks, oxygen and water reach rebar and cause deterioration.

Several other significant (and commonly seen) deterioration mechanisms are freeze and thaw deterioration, carbonation, chemical attack, alkali-silica reaction, abrasion, fire damage, and corrosion of dissimilar metals.

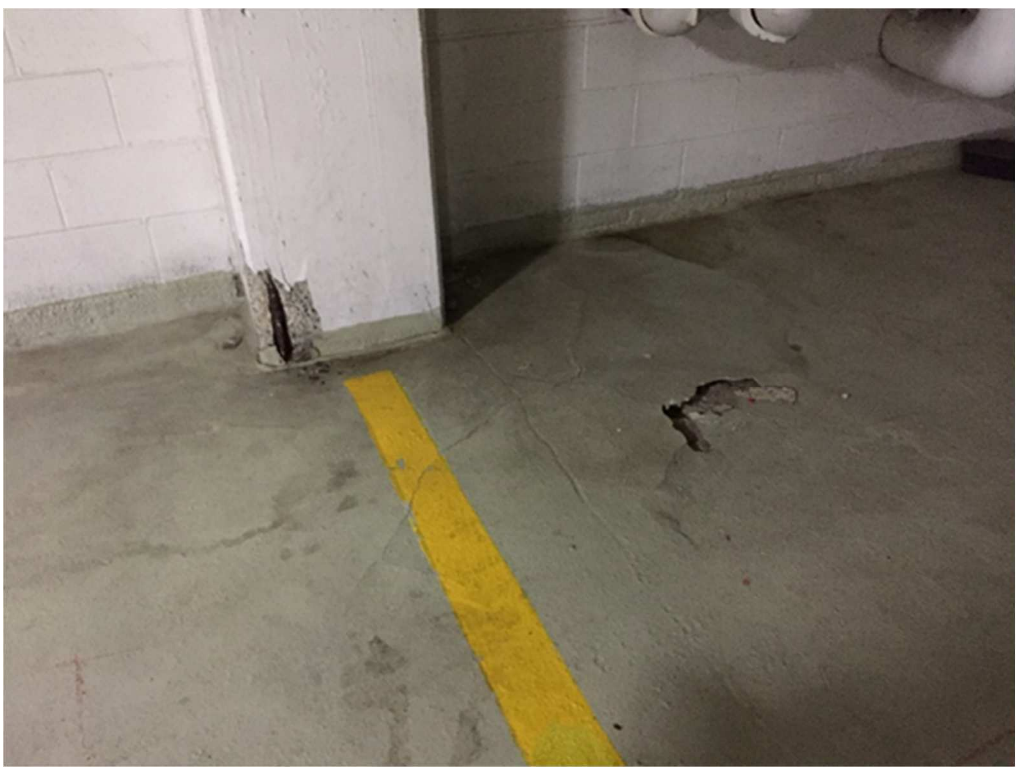


Figure 2-2-Slab Deterioration and Exposed Column Reinforcement

2.1.1 Carbonation

The simple chemical process associated with carbonation is expressed as:

$Ca(OH)_2 + CO_2 ==> CaCO_3 + H_2O$

$CaCO_3$ reduces the pH of concrete and affect the passivating layer around rebar, similar to chloride ions. According to Hanson (*2015*) and ACI 201 (*2008*), the rate of carbonation is optimum between 50% and 75% relative humidity; below 25%, the rate of carbonation is considerably low and above 75%, moisture in blocks carbon dioxide penetration into the concrete. Carbonation significantly reduces the number of chloride ions needed to initiate corrosion, which causes an exponential increase in deterioration in low-density concrete exposed to elements.

2.1.2 Chemical Attack

Chemical attack due to sulfates, acids, various alkali chemicals change the chemistry of the concrete and cause deterioration. Generally, soils containing aggressive chemicals or structures used to store or manufacture such chemicals need to be designed with concrete mixtures suitable to resist such deterioration. Discussion of every kind of chemical attack and their mechanisms are beyond the intent and scope of this research; however, several important chemical deterioration mechanisms as elaborated by ACI 201R.2R-08 are as follows as they relate to the topic:

2.1.2.1 Acid Attack: Reaction between acids and calcium hydroxide of the hydrated Portland cement forms water-soluble calcium compounds that leach out of the concrete over time. Acids also reduce the alkalinity and create a corrosive environment for the reinforcement.

2.1.2.2 Sulfate Attack: Most commonly observed byproducts of sulfate attack are ettringite and gypsum, which increase the volume of the solids within the matrix and cause cracks (similar to corrosion of the steel). When sulfates react with the hydrated cement, chemical reactions take place, and such chemicals form. Ettringite and gypsum also cause softening and loss of compressive strength, which significantly affect concrete columns.

2.1.2.3 Alkali-Silica Reaction (ASR): While aggregates do generally not react with cement, certain types of aggregates containing amorphous silica react with highly alkaline cement, which causes the formation of an expansive silica gel. Silica gel expands in the presence of moisture and causes the concrete to spall. While most types of chemical attack/reaction can be slowed down or remediated, ASR cannot be repaired. Sections of concrete containing ASR needs to be removed and replaced in their entirety. This issue is sometimes referred to as concrete cancer due to its invasiveness and lack of alternates to repair this type of damage.

2.1.3 Abrasion and Impact

Deterioration of the concrete cover over the reinforcement due to mechanical abrasion or vehicular impact is generally seen in industrial and parking structures. Unintentional impact-related damage is generally caused by equipment/vehicles moving at low speeds, which don't carry sufficient energy to cause deformations in the concrete columns. International Code Council (*2015*) requires parking structures to be designed to resist the impact loading imposed by vehicles; however, it does not specify what level of sustained damage is permissible after the impact. The apparent issue related to the loss of cover by mechanical abrasion; is commonly addressed by the use of dense concrete mixtures with abrasion-resistant aggregates.

2.1.4 Fires

Depending on the temperature of the concrete, varying degrees of damage is possible. While color change is observed at up to 550 ºF, at higher temperatures, concrete starts to crack and spall due to the evaporation of pore water and changes in the chemistry. Compressive strength significantly decreases, and aggregate displacement occurs at approximately 1,100 ºF. Depending on the depth of the fire damage, embedded reinforcement may also sustain damage (warping or spalling due to expansion). Fire damage assessment (when the temperatures reach approximately 1100 ºF) often requires cores to be taken and petrographically assessed to understand the depth of fire damage and extent of concrete deterioration

2.2 Repairing Damage

2.2.1 Removal and Replacement of Deteriorated Concrete and Reinforcement

All of the issues and background information was given to explain why concrete repairs would be necessary and give an idea about how they can be performed. Damaged sections of concrete with various types of damage are removed by jackhammering or hydro blasting and replaced with new concrete. As explained in Chapter 1, shear friction and bond strength are two critical components in repair. If demolition is not done correctly, the surface of the concrete will contain cracks, chips, and weak sections, which will reduce the bond as well as shear friction capacity. Hydrodemolition or water blasting is a relatively new method that utilizes high-pressure water, up to 35,000 psi, to remove the concrete with minimal fractures around the area being demolished. Both hydro demolition and jackhammering have their pros and cons but jackhammering with a 15 lb. electric hammer is the most common method of concrete demolition at concrete columns if the damage isolated (rather than demolition of the whole column in the case of an irreparable ASR damage). Equipment used in the hydro demolition method is best suited for slab applications where the water jet can be safely applied with water blasting robot vertically.

In the jackhammering method, perimeters of the repair areas are saw cut up to 1/8" to define the boundaries of the patches (and prevent surface cracking). After the deteriorated concrete is removed with a 15-lb jackhammer, the final part of the demolition, to remove rust and remaining concrete around the deformations, is made by abrasive blasting with a wire wheel or sandblasting. Repair surfaces are commonly jackhammered to obtain vertical patch surfaces with uniform cross-sections. Considering that small and left-over sections of the concrete can break and fail during demolition, this method is the most logical approach to attain uniform load transfer along with the repaired locations (**Figure 2-3**).

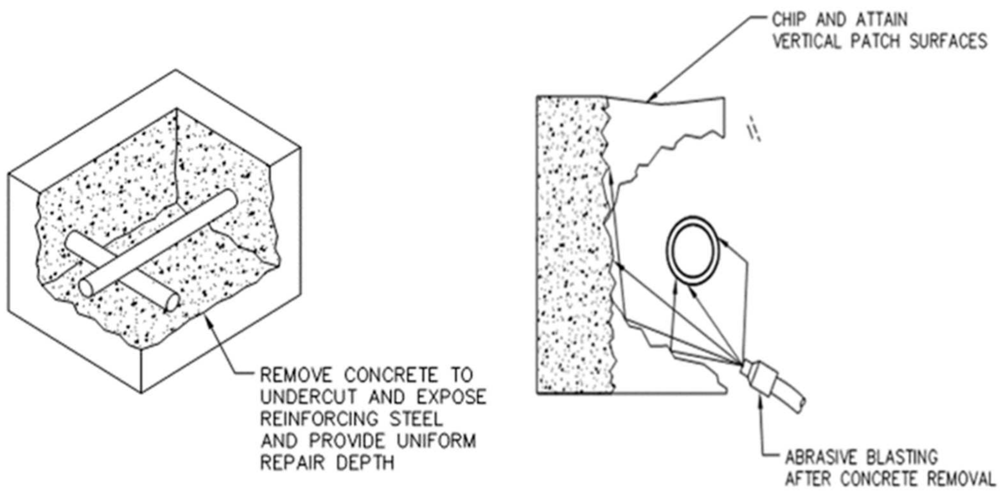


Figure 2-3-Concrete Removal for Repair

After the reinforcement is exposed and cleaned, observations need to be made to determine whether the section loss is significant enough to arrest the placement of additional reinforcement. Most practicing engineers limit the sectional loss to 15% by default; however, this arbitrary number may or may not be adequate depending on the location of the rebar and applied forces. Rebar couplers can also be utilized instead of lap splicing the bars; however, it is unlikely that such an approach would be practical in columns at typical building applications where the required cover would not be sufficient (unless the column is jacketed).

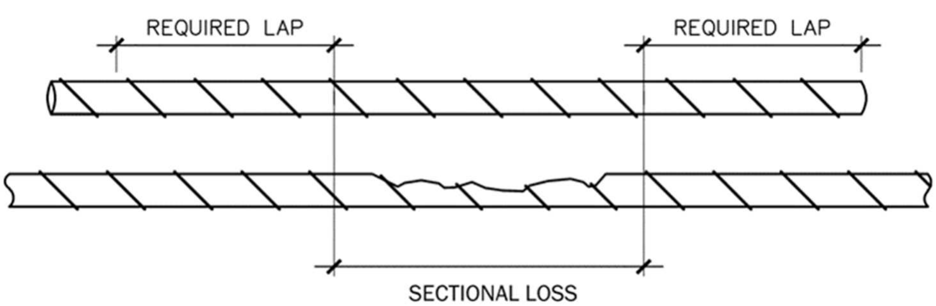


Figure 2-4-Lap Splice for Reinforcement

In flexural members, splice/lap length is determined by Section 25.5 of ACI 318 (*2014*) for tension splices and compression splices. If there is significant corrosion in the repaired section of the column, where the rebar needs to be spliced to compensate for the sectional loss, it may be more efficient to enlarge the column or repair the entire column as splices may require additional (and labor-intensive) demolition beyond the spalled sections. In most column repair applications, where isolated concrete column repairs are performed, the cross-sectional loss is not a significant concern, and splicing would ordinarily not be necessary.

Placement of concrete can be performed by forming and pouring (**Figure 2-7**), forming and pumping (**Figure 2-5**), or troweling the material to the prepared area (**Figure 2-6**). If forming and pumping can be accomplished, this method would yield the best results as high pressure within the formwork would promote a better bond between the repair material and the substrate. Similarly, applying the material with a trowel in multiple lifts would allow a strong bond between the repair material and the substrate; however, multiple lifts require the repair material to bond to itself several times, and there is a higher probability of a bond failure due to workmanship and material issues.

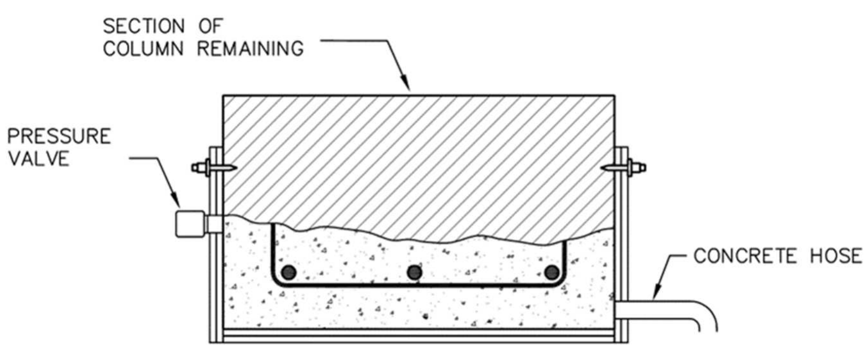


Figure 2-5-Form and Pump Repair Column Section

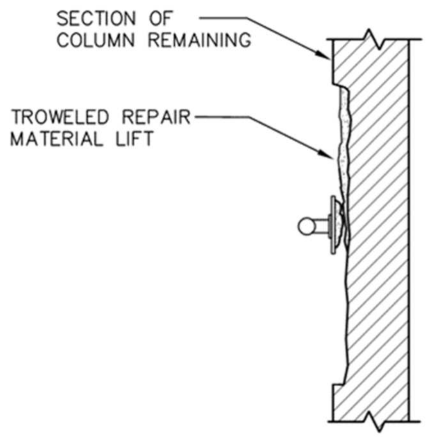


Figure 2-6-Trowel Repaired Column Elevation

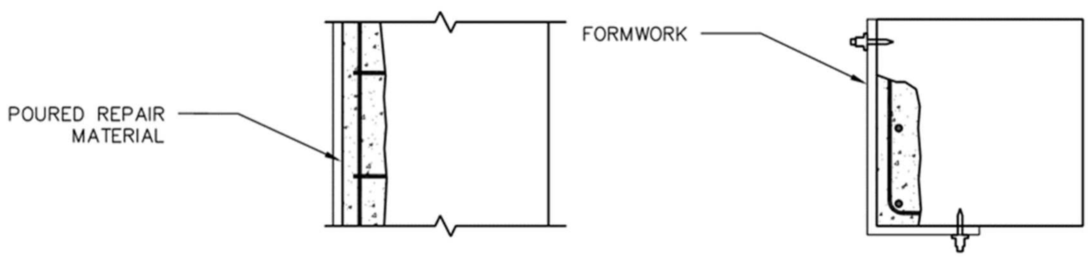


Figure 2-7-Form and Pour Repair (Section and Elevation)

If the concrete is only cracked and the section did not debond from the surrounding concrete or the rebar, epoxy crack injection methods (low pressure or high pressure) are utilized to stitch the crack surfaces together. If the cracks are deep (such as drying shrinkage or structural cracks associated with shear or flexure), the high-pressure injection method would be required to penetrate through the entire crack depth (**Figure 2-8**) fully. Pressure injection is performed with widely available pumping equipment and plastic injection ports anchored into the concrete along the crack. As the epoxy is injected, each port is inspected to observe the flow of the material and capped once the material oozes out of the port.

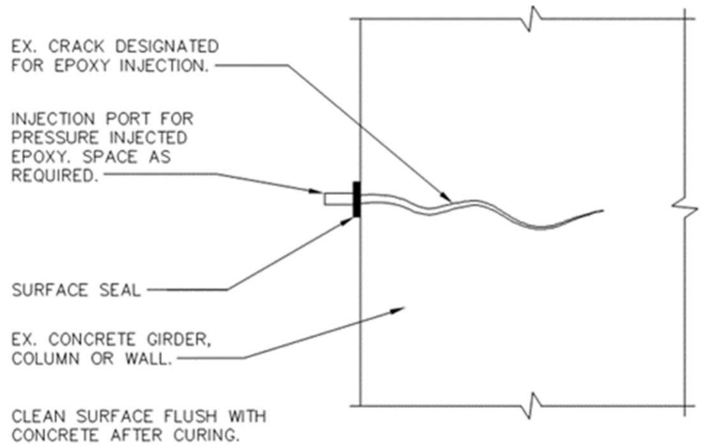


Figure 2-8-Crack Injection with Epoxy (Courtesy of Ehlert-Bryan Inc.)

Modulus of elasticity and viscosity are two of the important material properties needed to be considered in the injection of column cracks. Generally, high modulus epoxies are used in applications where minimal movement is expected as such materials tend to crack due to their stiffness. Low modulus epoxies are used in flexural members where deformation is expected during service. Low viscosity epoxies are suitable to be pressure injected into cracks due to their fluidity; high viscosity epoxies (with gel consistency) are used to fill shallow cracks as their injection cannot be performed with standard pressure injection equipment.

Chapter 3: Utilized Repair Methods and Load Testing

3.1 Column Repairs

In order to replicate typical shallow and deep column repairs, nine columns were repaired 28-days after they were cast. Repairs were performed at 6” x12” (Height x Arc Length) areas at the bases of the columns at varying depths (depending on the type of repairs). Repairs were not extended to the entire perimeter of the columns to find out whether or not having a joint between the stronger repair material and weak existing concrete would cause any localized stress concentrations in the area surrounding a patch.

At shallow repairs (labeled as SR1 through SR3), concrete demolition was extended up to the exterior face of the vertical reinforcement (**Figure 1-2**). Before the removal of the concrete, perimeters of the patches were saw-cut to 1/8” depth to prevent cracks around the demolition area as it is commonly done by utilizing the jackhammering method.

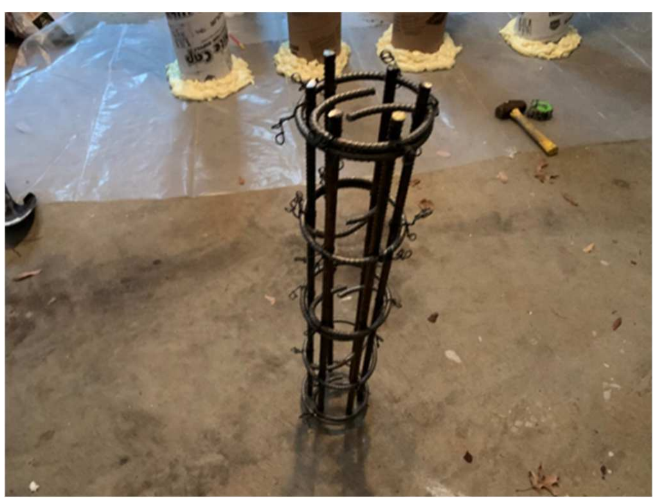


Figure 3-1-Column Rebar Cage

ACI 318 *(2014)*'s detailing guidelines followed for the column samples to avoid local buckling of the rebar; thus, (6) vertical #4 bars were placed around #3 circular column ties (with standard hooks) which were hooked to alternating vertical bars to prevent cracks from forming at the surface of the columns during the load test. A regular weight, commercially available bag mix with ¾" aggregate was chosen due to its low-cost and availability as opposed to a higher strength and self-consolidating mix. As seen in **Figures 3-2 and 3-3** below, partial and full depth concrete repairs were performed at the side of the column where the vertical bars were extended to the horizontal face of the columns to prevent potential issues with cracks/fissures at the which could have occurred chipping the opposite end with more concrete cover. During the demolition of the concrete, at sections of the columns where the aggregate became loose or the concrete fracted beyond the 6"x12" areas, repairs were extended to such areas to ensure that no loose concrete was left within the section of the concrete being repaired. Total repair areas are summarized in **Table 1** below.

Figure 3-2-Demolition at Sample SR3

Sample ID	Repair Type	Shallow Repair Area (in^2)	Shallow Repair Average Depth (in)	Deep Repair Area (in^2)	Deep Repair Average Depth
SR1	Shallow	72	0.6	0	0
SR2	Shallow	79.5	0.6	0	0
SR3	Shallow	93	0.6	0	0
DR1	Deep	21	0.6	72	1.75
DR2	Deep	7	0.6	72	2
DR3	Deep	3	0.6	72	2

Table 1-Repair Areas at Each Sample

While concrete has excellent durability, its structural properties vary due to the proportioning of materials within the mix. Materials' locations within the mix and variability of the materials' properties require the use of at least (3) samples to attain a reliable result for testing. For the very same reason, most design codes require the concrete compressive strength and other structural properties to be determined by taking the average of (3) tests. Following the same methodology, repairs were performed at (3) samples of each type. As seen in **Table 1** above, each column repair had slight variations in terms of repair areas, as each column section showed variations.

Figure 3-3-Demoliton at Sample DR2

As for the surface roughness, each horizontal and vertical column surface was roughened with a chisel tipped electric hammer to ¼" magnitude to attain the highest shear friction resistance between the new and existing concrete possible. Repair surfaces were detailed to be uniform and straight. Demolished surfaces were visually inspected to ensure that there are no chips, spalls, cracks, or loose sections of concrete. Prior to testing the columns, levelness of the top and bottom sections of the samples were checked, and, as necessary, an approximately 8,000-psi leveling material (hydrocal) was placed at up to 1/4" thickness to ensure that the top and bottoms of the columns were completely flat.

Concrete repairs were performed with high performance, polymer-modified, shrinkage compensated self-leveling repair mortar with a 28-day compressive strength of 5,600 psi (Appendix). After the repair surfaces were scrubbed clean, they were brought to SSD (surface saturated dry) condition (with no standing water visible), and repair material was placed by forming and pouring into the prepared patch areas. Concrete repairs were left to cure for 28 days at 65 °F and 45% relative humidity before being load tested. This material was selected to ensure that proper consolidation can be attained within the small patch areas which cannot be vibrated, shrinkage cracks can be avoided, and there is not an extreme difference in the modulus of elasticity. Furthermore, the chosen repair material is commonly used in applications where resistance is required against chloride penetration.

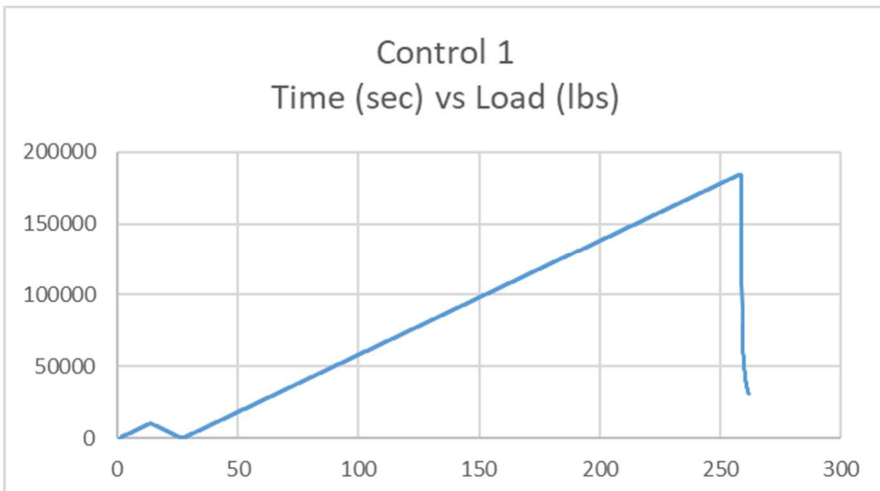
Before testing, small plastic shrinkage cracks (less than 2" long and ¼" deep) were sealed by utilizing a high modulus and viscosity crack repair epoxy to ensure that such cracks do not open up during the load test and alter the results of the testing. Epoxy was allowed to cure before load testing.

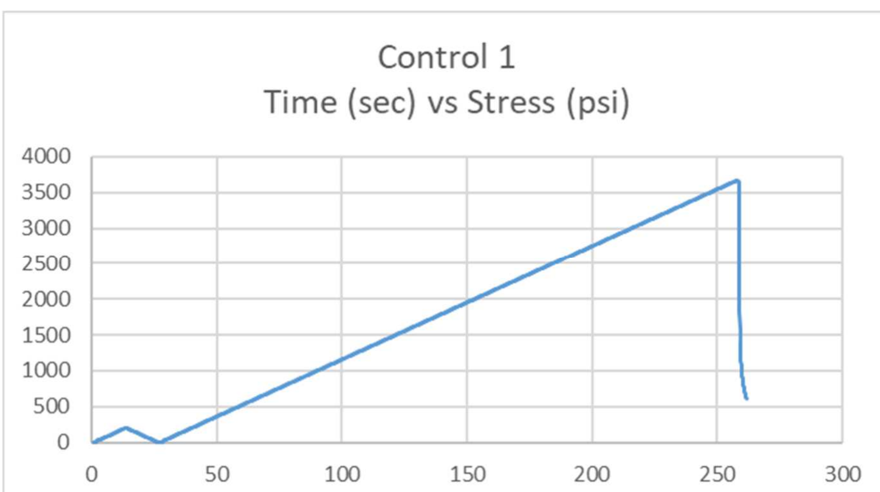
Axial load testing was performed with a 400-kip capacity universal load testing machine, as seen in **Figure 3-4** below. Since the load testing equipment was capable of inducing moment to the column, top and bottom surfaces of the column samples were flattened to prevent accidental eccentricities.

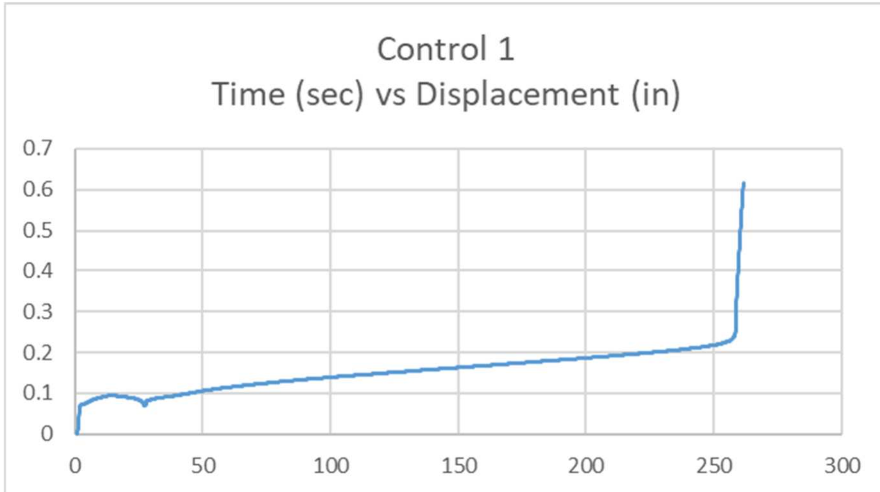
Figure 3-4- Axial Load Testing Equipment

In order to ensure complete contact at the bottom of the testing machine, a ½" thick steel plate was placed to fill in gaps between the steel slats at the bottom of the columns.

After the columns were secured to the testing machine, and it was ensured that the sample was centered to avoid eccentric loading, axial loading was applied in 2 stages. During stage 1 or "preloading stage," columns were loaded up to 10,000 lbs and then unloaded to allow the columns to engage the entire column section. Following the preloading stage, columns were loaded to failure. The figures below contain the results of the load testing at each column:



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Time (sec) vs Displacement (in)
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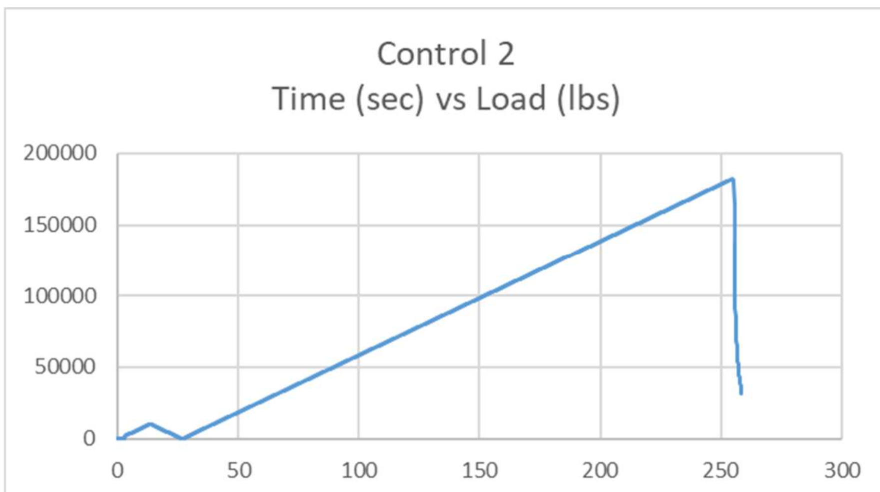
0.6
0.5
0.4
0.3
0.2
0.1
0
0
50
100
150
200
250
300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Time (sec) vs Load (lbs)
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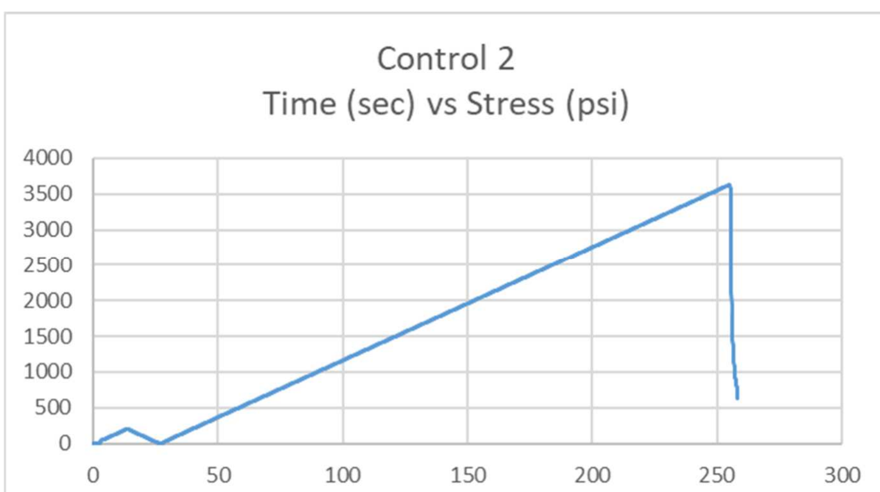
100000
50000
0
0
50
100
150
200
250
300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Time (sec) vs Stress (psi)
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2500
2000
1500
1000
500
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0
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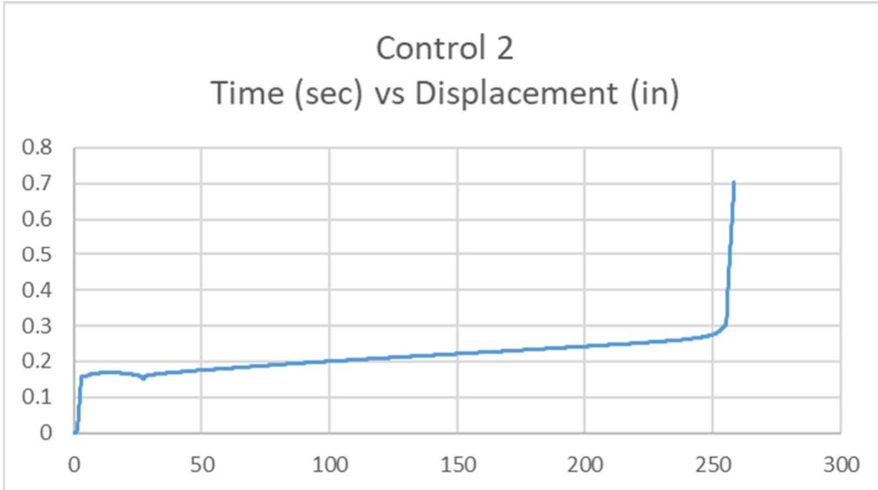
100
150
200
250
300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Time (sec) vs Displacement (in)
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0.5
0.4
0.3
0.2
0.1
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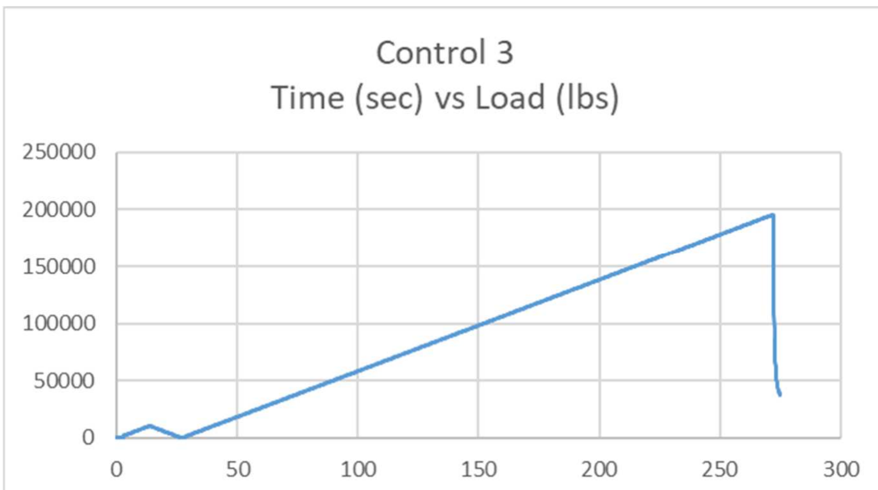
100
150
200
250
300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100000
50000
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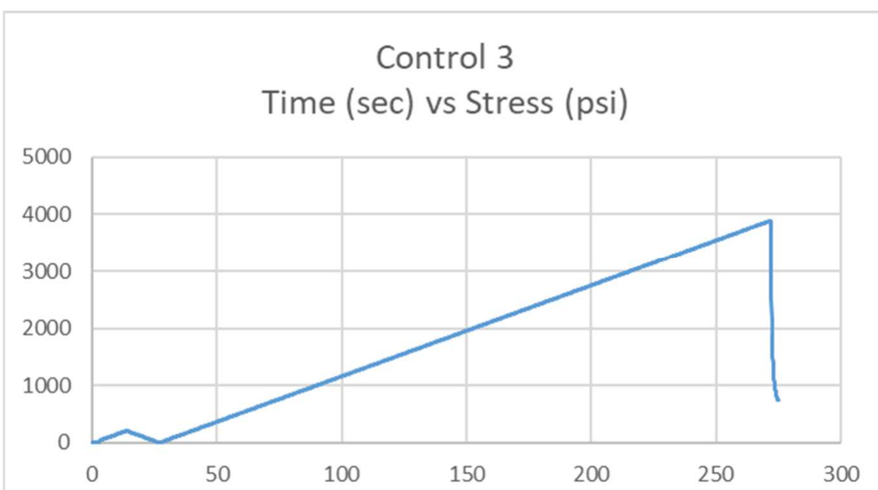
0
50
100
150
200
250
300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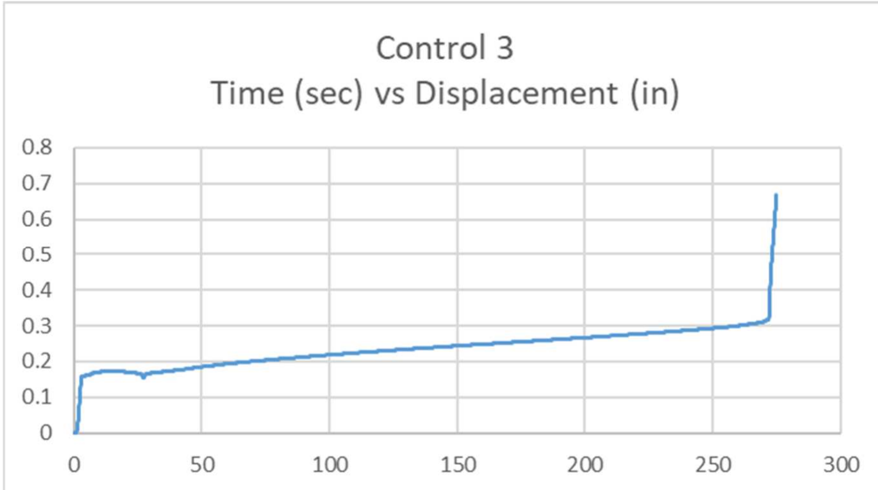
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Time (sec) vs Displacement (in)
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0.4
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200
250
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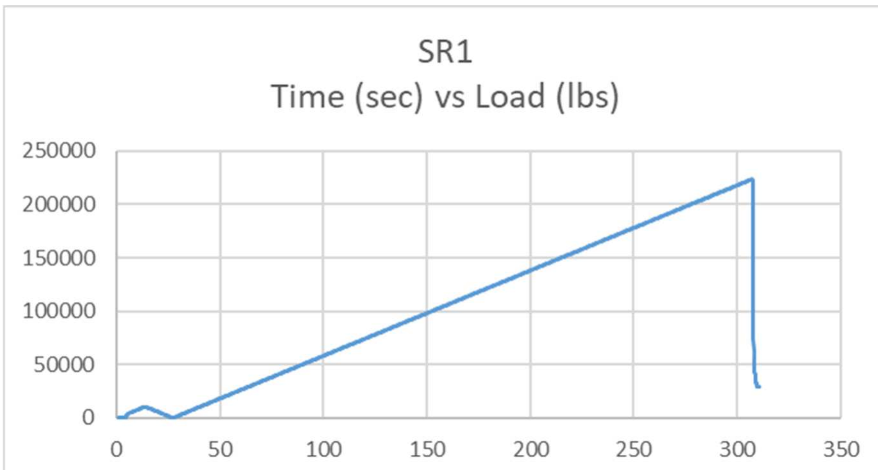
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Time (sec) vs Load (lbs)
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150000
100000
50000
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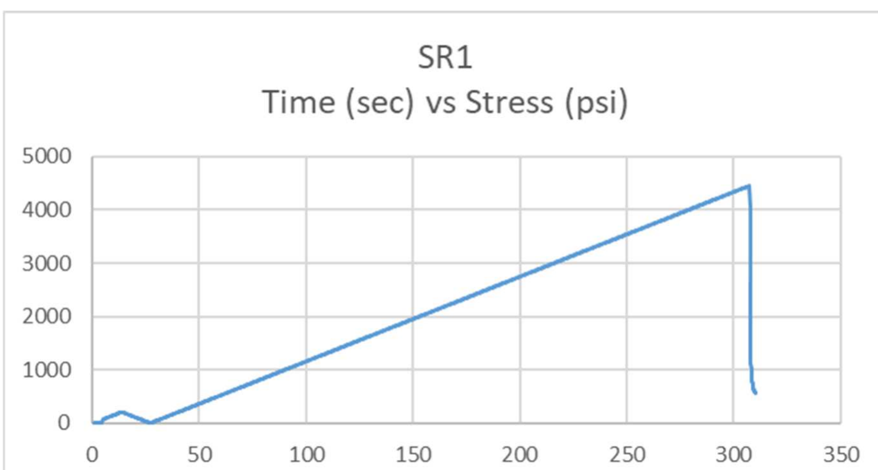
0
50
100
150
200
250
300
350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Time (sec) vs Stress (psi)
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2000
1000
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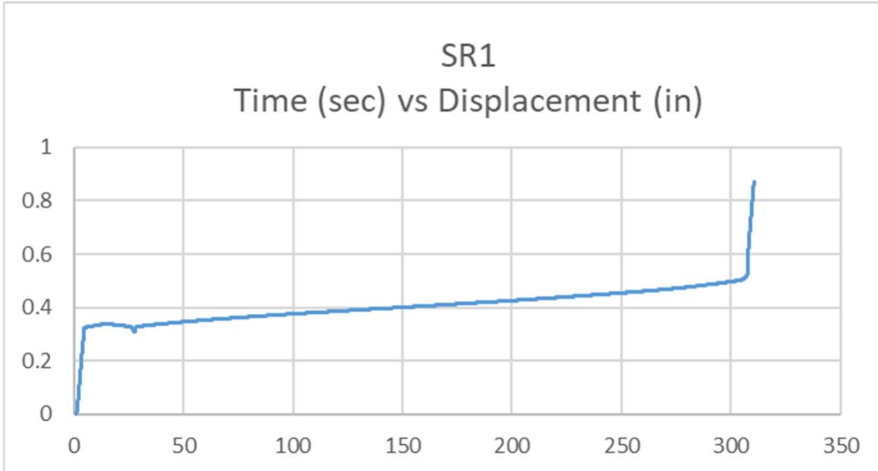
50
100
150
200
250
300
350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Time (sec) vs Displacement (in)
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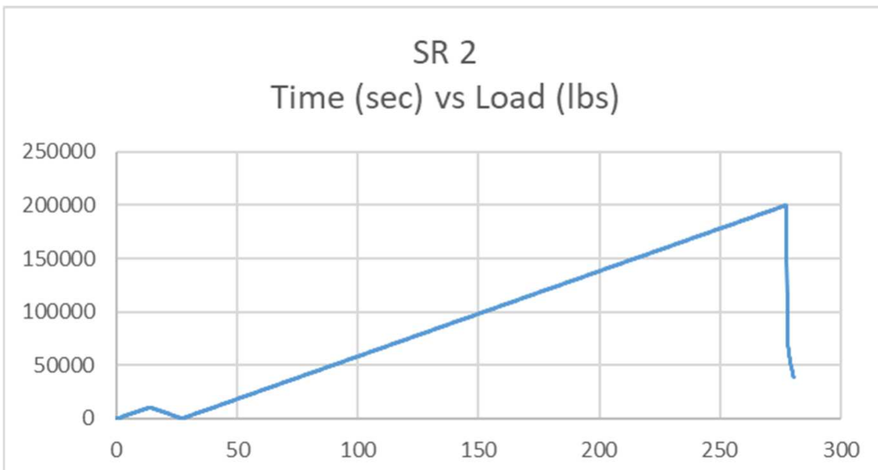
250
300
350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Time (sec) vs Load (lbs)
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100
150
200
250
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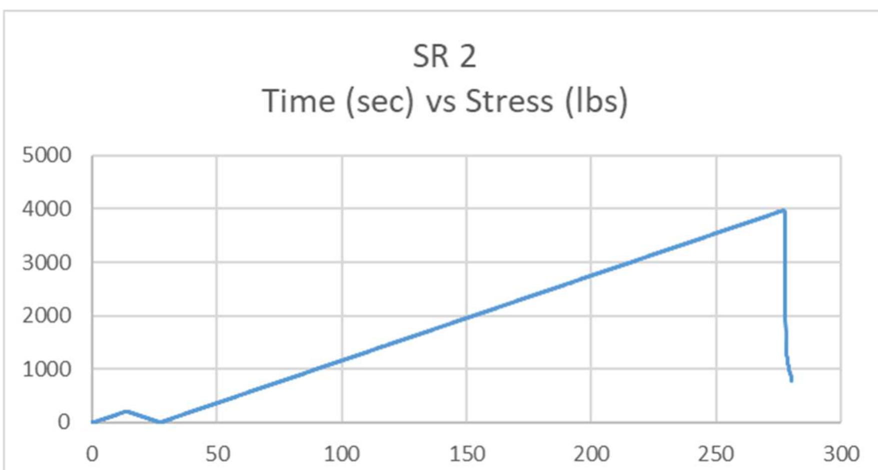
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Time (sec) vs Stress (lbs)
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4000
3000
2000
1000
0
0
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250
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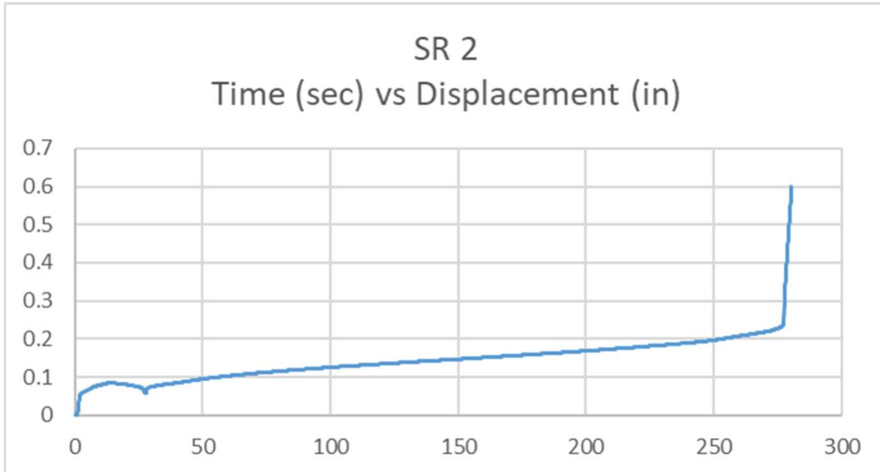
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Time (sec) vs Displacement (in)
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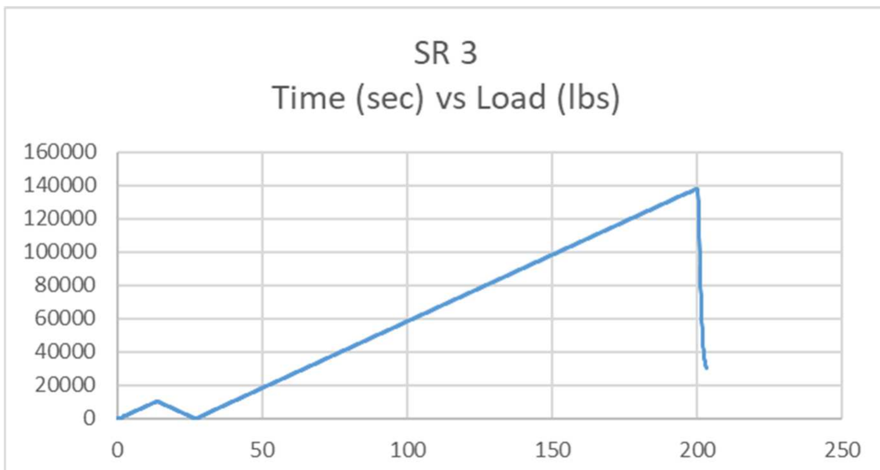
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Time (sec) vs Load (lbs)
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60000
40000
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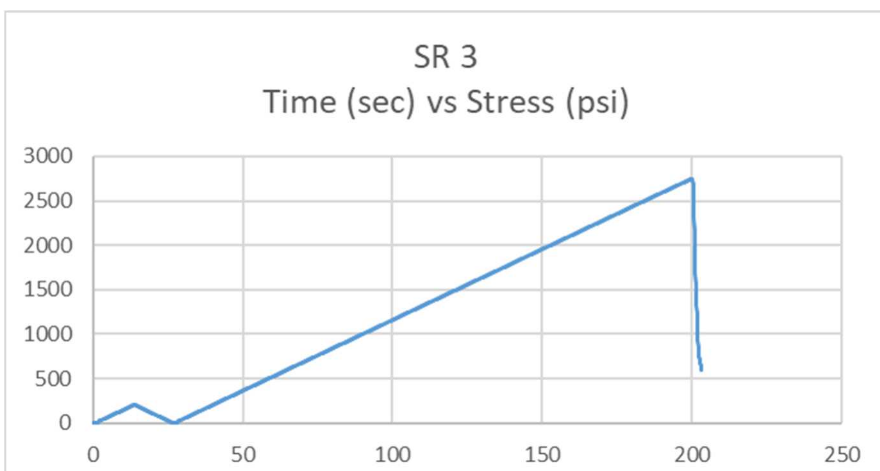
50
100
150
200
250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Time (sec) vs Stress (psi)
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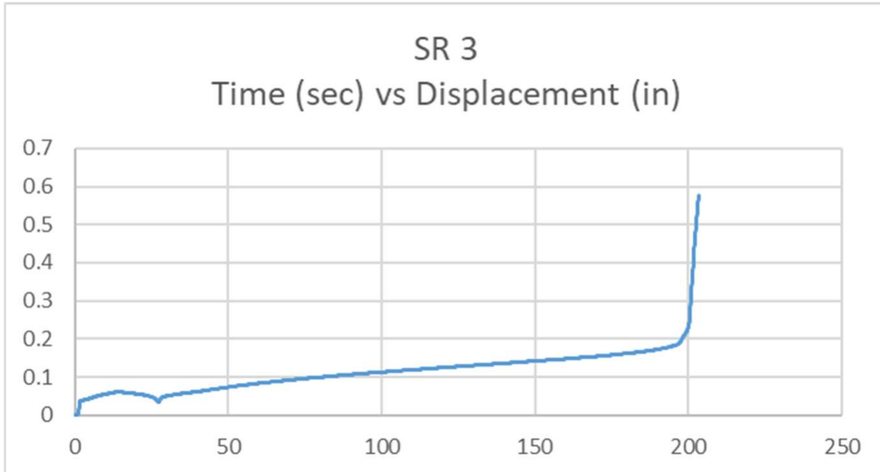
2500
2000
1500
1000
500
0
0
50
100
150
200
250


SR 3
Time (sec) vs Displacement (in)
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0.3
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0.1
0
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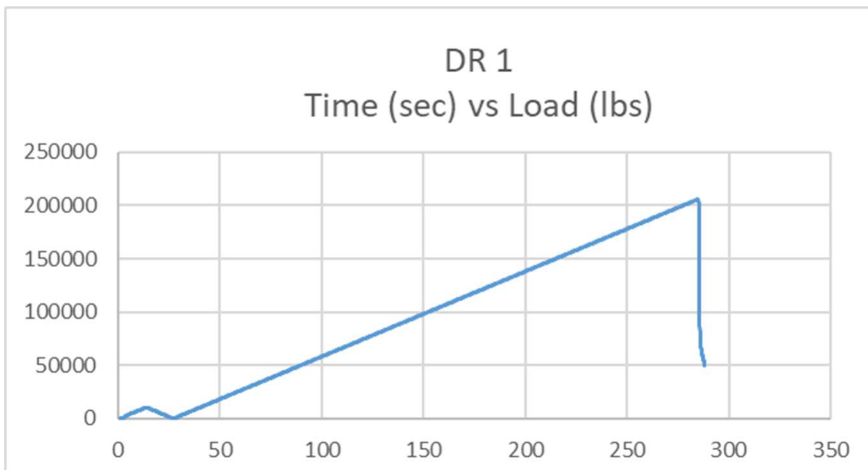
150
200
250


DR 1
Time (sec) vs Load (lbs)
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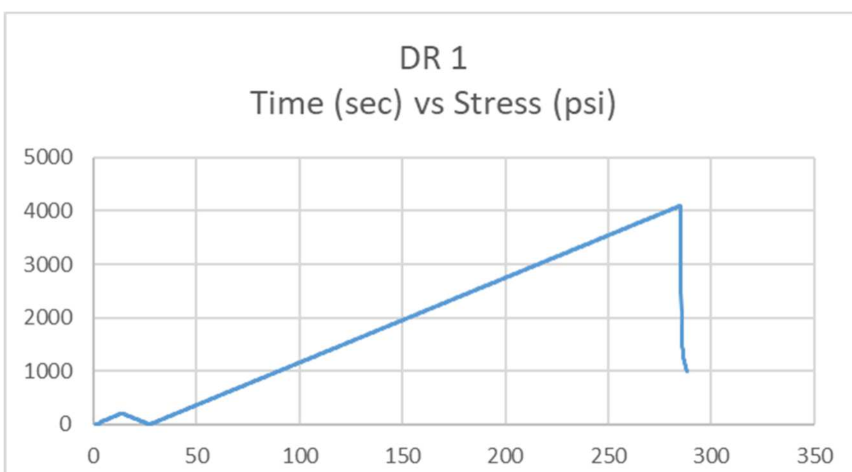
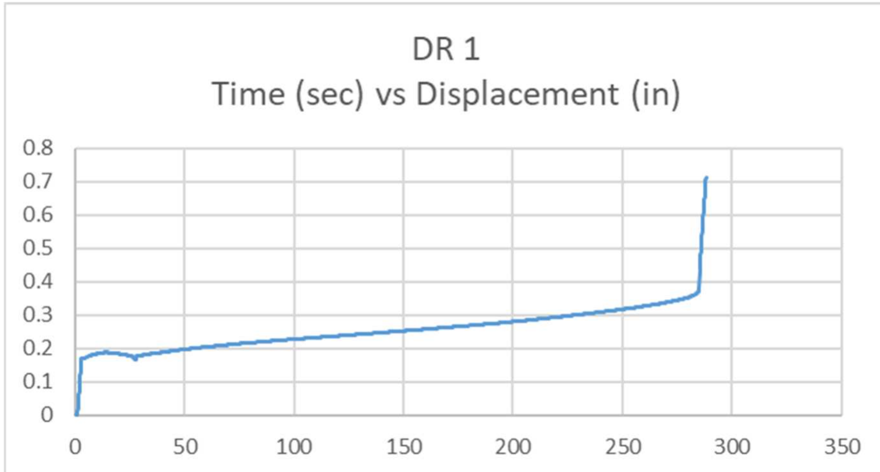
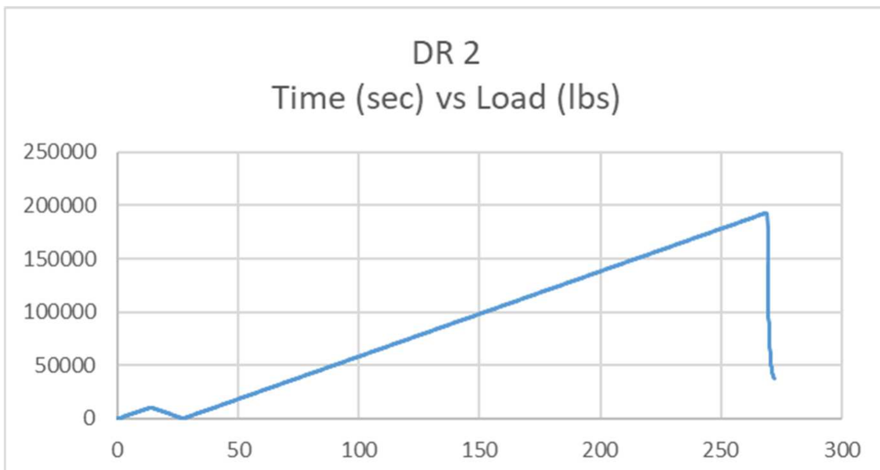
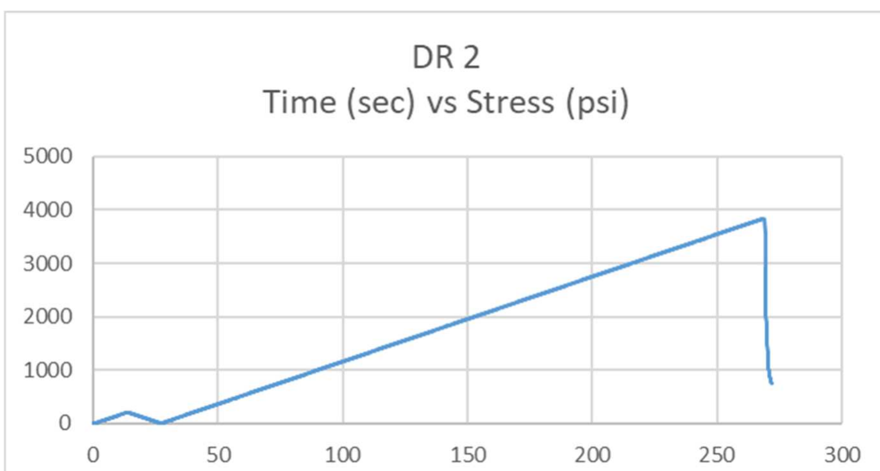
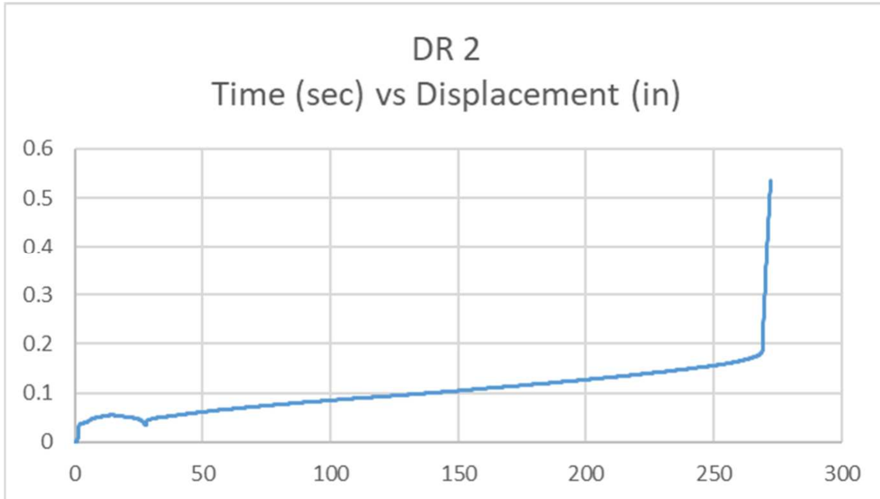
150000
100000
50000
0
0
50
100
150
200
250
300
350

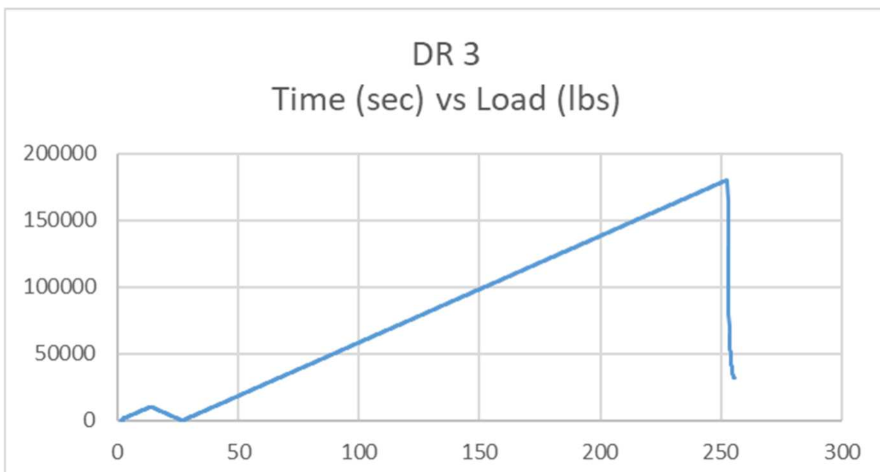

DR 1
Time (sec) vs Stress (psi)
5000
4000
3000
2000
1000
0
0
50
100
150
200
250
300
350

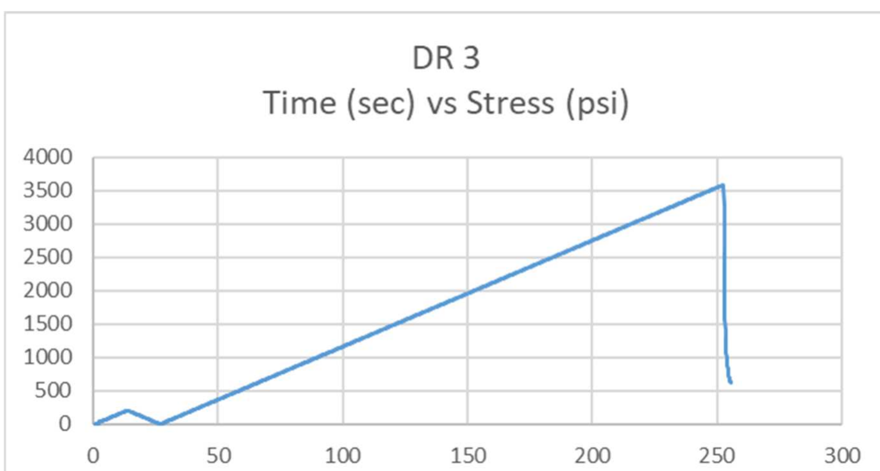

DR 1
Time (sec) vs Displacement (in)
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
50
100
150
200
250
300
350


DR 2
Time (sec) vs Load (lbs)
250000
200000
150000
100000
50000
0
0
50
100
150
200
250
300


DR 2
Time (sec) vs Stress (psi)
5000
4000
3000
2000
1000
0
0
50
100
150
200
250
300


DR 2
Time (sec) vs Displacement (in)
0.6
0.5
0.4
0.3
0.2
0.1
0
0
50
100
150
200
250
300


DR 3
Time (sec) vs Load (lbs)
200000
150000
100000
50000
0
0
50
100
150
200
250
300


DR 3
Time (sec) vs Stress (psi)
4000
3500
3000
2500
2000
1500
1000
500
0
0
50
100
150
200
250
300

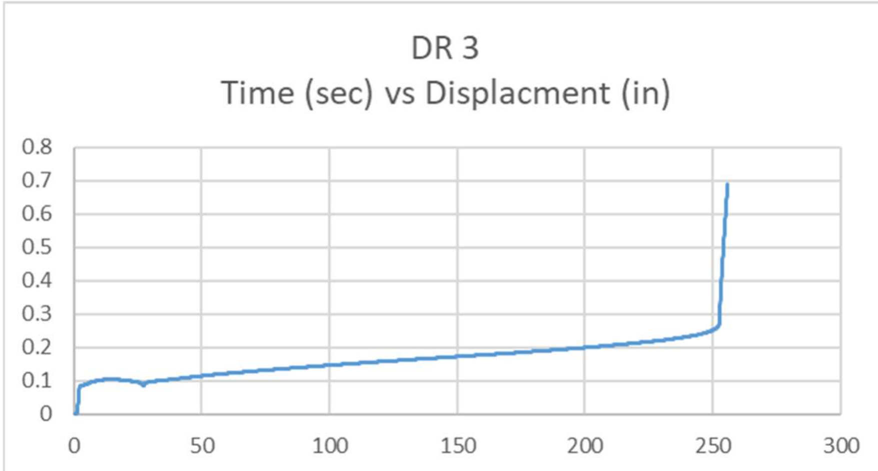


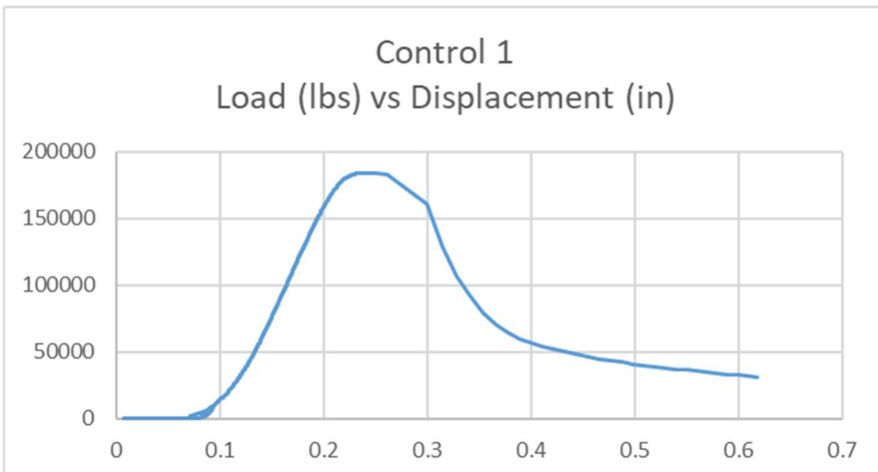
Figure 3-5-Test Results for all Columns

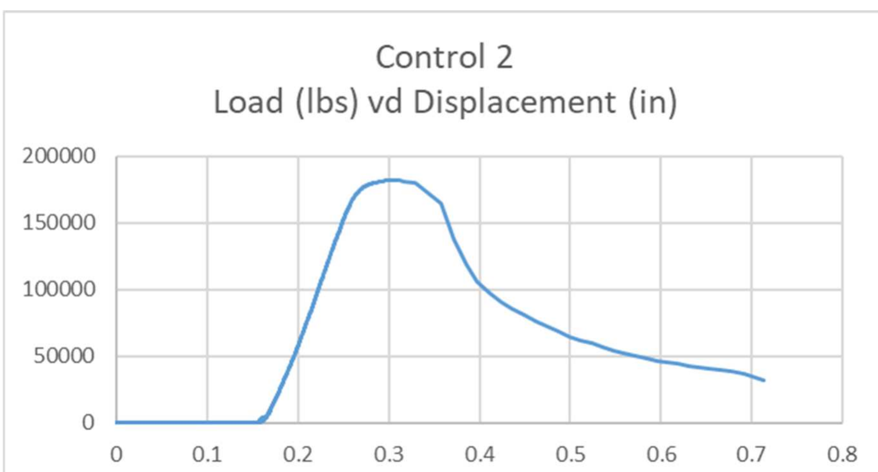
As seen above, all columns behaved similarly and, with the exception of SR3, obtained maximum load and deformation values were close to each other. Based on the obtained results, the following are the maximum and average load, compressive stress, deformation and modulus of elasticity:

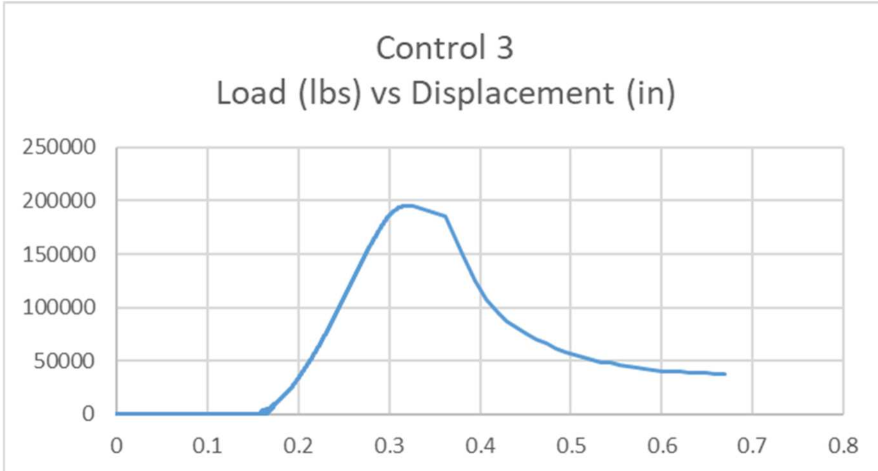
Sample ID	**Max. Load (lbs)**	**Max. Stress (psi)**	**Initial Length (in)**	**Max. Disp. (in)**	**Strain**	**Dia. (in)**	**Area (in^2)**
Control 1	184,423.00	3,668.98	36.00	0.234	0.007	8.00	50.24
Control 2	182,128.00	3,623.32	36.50	0.300	0.008	8.00	50.24
Control 3	195,478.00	3,888.91	36.75	0.319	0.009	8.00	50.24
SR1	223,444.00	4,445.28	36.19	0.520	0.014	7.92	49.24
SR2	199,607.00	3,971.00	36.50	0.236	0.006	8.00	50.24
SR3	138,098.00	2,747.37	37.00	0.230	0.006	7.80	47.76
DR1	205,701.00	4,029.29	36.19	0.373	0.010	8.00	50.24
DR2	192,981.00	3,839.23	36.00	0.187	0.005	8.00	50.24
DR3	180,075.00	3,582.48	36.25	0.269	0.007	8.10	51.50
Ave. for DR	192,919.00	3,817.00	36.15	0.276	0.008	8.03	50.66
Ave for SR	187,049.67	3,721.22	36.56	0.329	0.009	7.91	49.08
Ave for Control	187,343.00	3,727.07	36.42	0.284	0.008	8.00	50.24

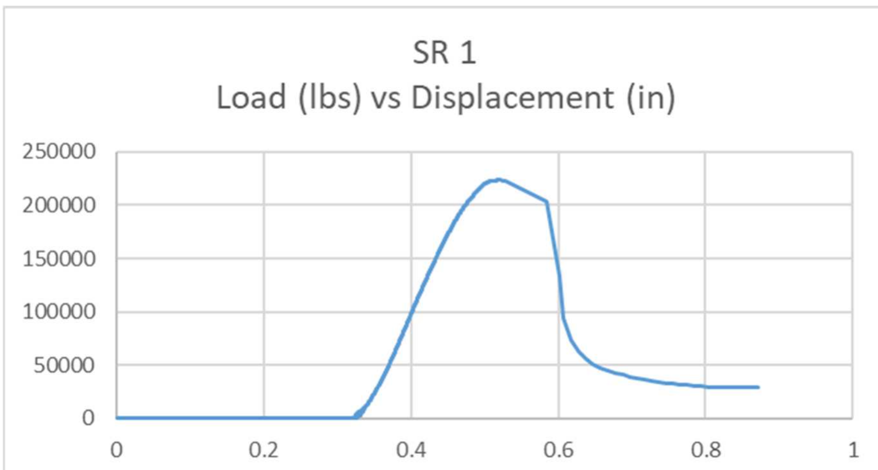
Table 2-Observed Maximum Values

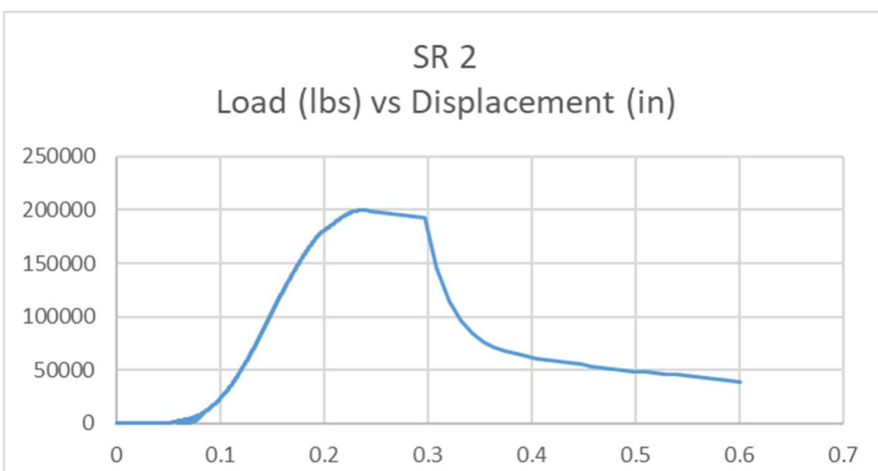
Results of the load tests yield very close axial capacity values and are revealing. It can be seen that the capacity of the columns which underwent shallow repairs is almost identical to control columns. Also, the capacity of the columns, which underwent deep repairs, increased by approximately 3%. The increase in the capacity of the deep repairs is relatively small and discussed in the following chapter. The relationship between the compressive load, along with load vs. displacement values are as follows:

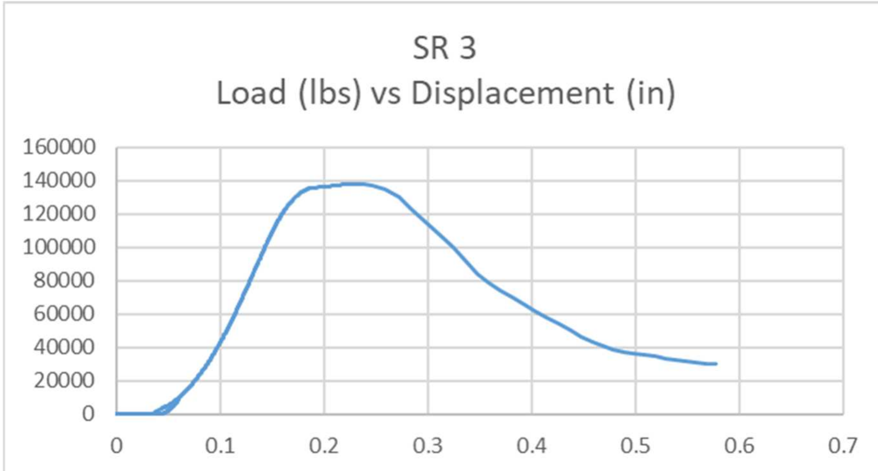


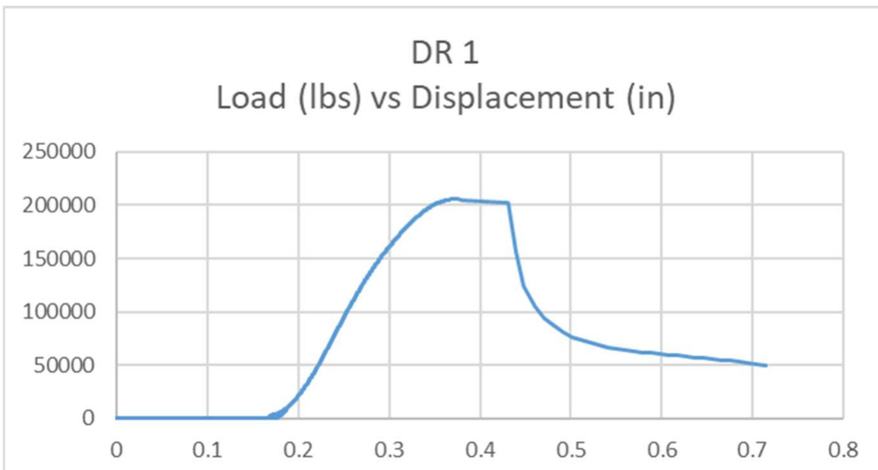


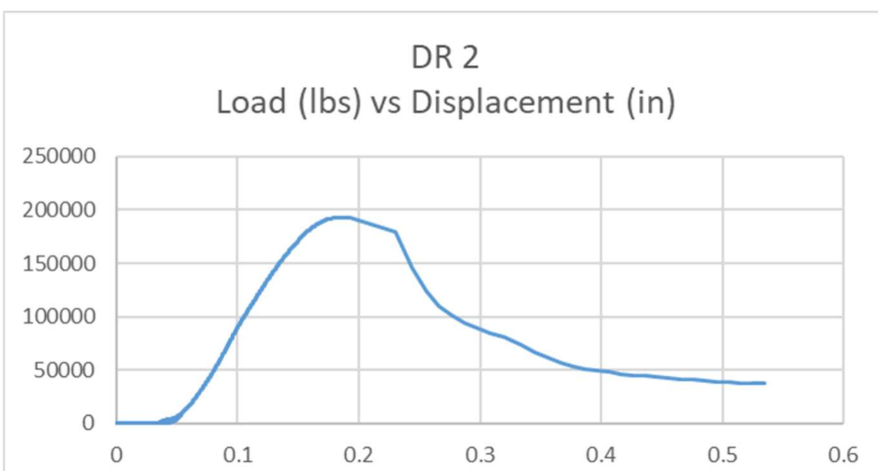

Control 3
Load (lbs) vs Displacement (in)
250000
200000
150000
100000
50000
0
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8


SR 1
Load (lbs) vs Displacement (in)
250000
200000
150000
100000
50000
0
0
0.2
0.4
0.6
0.8
1


SR 2
Load (lbs) vs Displacement (in)
250000
200000
150000
100000
50000
0
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7


SR 3
Load (lbs) vs Displacement (in)
160000
140000
120000
100000
80000
60000
40000
20000
0
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7


DR 1
Load (lbs) vs Displacement (in)
250000
200000
150000
100000
50000
0
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8


DR 2
Load (lbs) vs Displacement (in)
250000
200000
150000
100000
50000
0
0
0.1
0.2
0.3
0.4
0.5
0.6

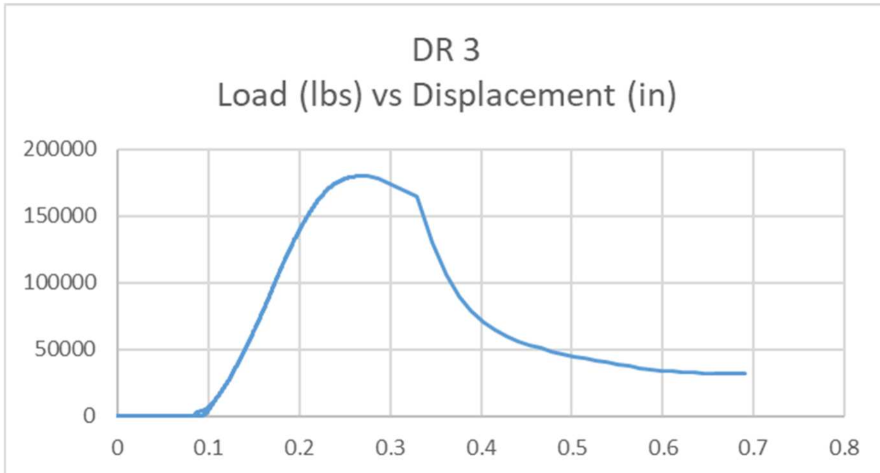


Figure 3-6-Load vs Displacement Values for All Columns

At all column samples except for sample SR3, a distinctive V-shaped compressive cone failure pattern was observed when the columns were tested. At all columns, the failure initiated at the ends of the columns where the rebar was not extended to the surface. The following are the photographs of the columns after failure:

Figure 3-7-Control 3 after the Load Test

Figure 3-8-Control 2 after Load Test

Figure 3-9-Control 1 after the Load Test

Figure 3-10-DR3 after the Load Test

Figure 3-11-DR2 after the Load Test

Figure 3-12-DR1 after the Load Test

Figure 3-13-SR3 after the Load Test

Figure 3-14-SR2 after the Load Test

Figure 3-15-SR1 after the Load Test

The distinct compressive failure of the columns is an indication that the samples behaved as intended. Failure initiated around the rebar where it was bearing on the concrete and then caused the outer section of the columns to split in a pure compressive failure. Almost identical failure mechanisms were observed on control columns as well as the repaired samples.

Except for column sample DR3, none of the repaired columns exhibited any cracks or debonding. In the case of sample DR3, cracking and spalling was noted was observed around the patch which started to propagate through the unrepaired section of the column as seen in **Figure 3-16** below:

Figure 3-16-Sample DR3 with cracks at the Unrepaired Side

The highest capacity was obtained from sample SR1 with 223.44 kips, where the vertical rebars started to buckle due to bearing over the concrete section at failure. No other issues related to vertical rebar or column ties were noted at the remainder of the samples. Lowest capacity was obtained from sample SR3, where the most partial depth repairs were performed. In terms of total repair volume, most repairs were performed at column DR1, which exhibited the second highest compressive capacity with 205.7 kips.

In terms of axial deformation, control columns and columns with deep repairs had almost identical average deformation results (within 3% of each other). Columns with shallow repairs exhibited the most deformation, which was 15% greater than that of the control samples; but, the high average number for the shallow repair columns is affected by the fact that the highest capacity was obtained from sample SR1 with deformation of 0.52” at failure.

Chapter 4: Discussion of Test Results

The results of the load test provided very important results regarding the effectiveness of concrete repairs in compression members. First and foremost, the average compressive capacities obtained from all three types of repairs were extremely close. The average compressive capacity for control samples was 187.34 kips, shallow repairs were 187.05 kips, and deep repairs were 192.92 kips. Considering the fact that different types and amounts of repairs were performed at different depths, obtaining such close capacities is a clear indication that the shallow and deep repairs are allowing the repaired concrete to fully transfer the load and allowing the concrete to behave as it was designed. As elaborated previously, concrete is not necessarily an isotropic and homogenous material; concrete samples cast from the same mixture do not exhibit the same structural properties due to the distribution of aggregates, sand, cement, and admixtures. The complex structure of the concrete would not allow a concrete member to exhibit precisely the same properties throughout its cross-section. As the properties of each column were slightly different due to normal variances within the mix, using the average of (3) samples was necessary to account for such variations and obtain average values for different types of repairs performed.

When the results of the axial testing were compared with the repair surface area, it was determined that the capacity of the columns increased as the deep repair area increased as seen in **Figure 4-1**; conversely, axial capacity decreased as the shallow depth area increased as seen in **Figure 4-2.**

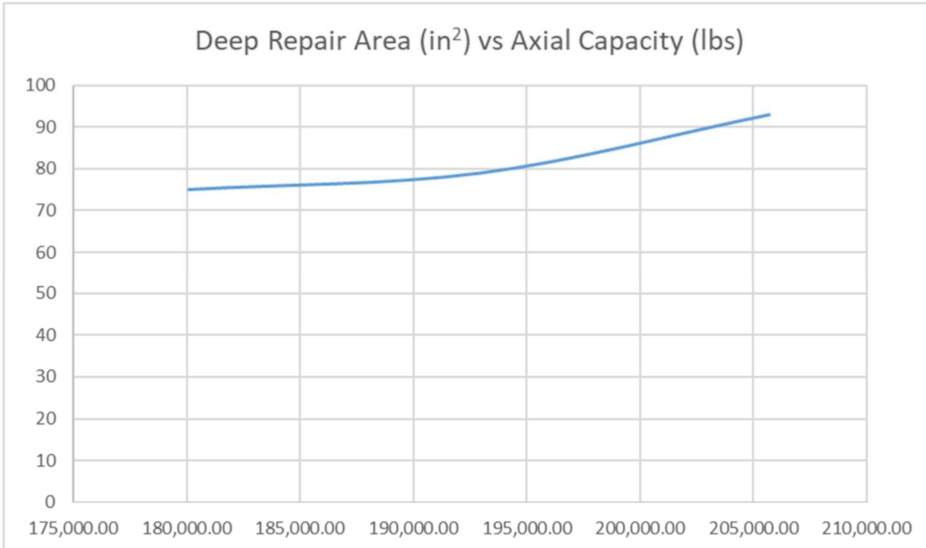


Figure 4-1-Deep Repair Area vs Axial Capacity

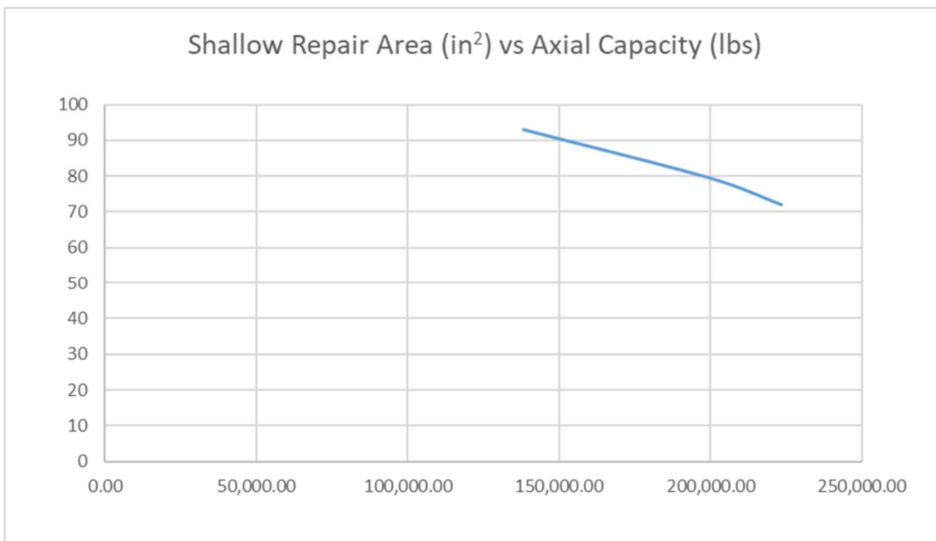


Figure 4-2-Shallow Repair Area vs Axial Capacity

While the increase in the column capacity makes sense, the decrease in the axial capacity with the increased shallow repair area can be explained by the variations in the material properties. Both the highest and lowest capacities were obtained from samples which have undergone shallow repairs, particularly having SR1 with the highest axial compressive capacity of 223.44 kips and lowest repair area of 72 in^2 skews the area vs capacity chart for shallow repairs. When the values are reviewed in light of the average values for the two different types of repairs and control samples (**Table 3**), it can be seen that the average values for the shallow repairs are almost identical to control samples. Hence, it can be concluded that the decrease in the axial capacity with increased shallow repair area is related to variations in the concrete properties.

Sample ID	Max. Load (lbs)	Max. Stress (psi)	Initial Length (in)	Max. Disp. (in)	Strain	Dia. (in)	Area (in^2)
Ave. for DR	192,919.00	3,817.00	36.15	0.276	0.008	8.03	50.66
Ave for SR	187,049.67	3,721.22	36.56	0.329	0.009	7.91	49.08
Ave for Control	187,343.00	3,727.07	36.42	0.284	0.008	8.00	50.24

Table 3-Average Values for Repairs and Control

Another important factor in the repair and sample preparation process was shrinkage. As explained in the preceding chapters, many issues leading to concrete spalls are initiated by shrinkage of the concrete. In order to get an accurate understanding of the behavior of the concrete repairs and minimize the effects of differential shrinkage, columns were allowed to cure for 28 days to enable them to go through their shrinkage cycle before any repairs were performed. Similarly, repairs were allowed to cure for 28 days to allow the repair material to shrink and impose shrinkage/creep stresses along the bond line. By considering the average strain values given in Chapter 1, the expected shrinkage deformation due to strain along the bond line is 0.000063 inches for a 6" tall concrete patch. Furthermore, the stress generated by creep would be:

$\mathcal{E} = 0.03\text{x } 10^{-6}$ strain/psi x 3,500 psi = 0.0001

$\tau_{creep} = \text{Ec} \text{x} \mathcal{E}$

$= 3.3\text{x}10^{6}$ psi x 0.0001 = 346.5 psi

While these values are somewhat high, they are only theoretical and do not represent reality due to the complexity of the repair geometries. Creep/shrinkage stresses cannot cause a patch material to debond from the substrate unless the repair material's tensile and bond capacities are exceeded. The reapplication of the compression load to the column will cause deformation in the concrete surrounding the repair material; furthermore, the shear friction capacity of the patch material will overcome the creep forces if they exceed the materials tensile stress capacity. The repair material used during the testing had a tensile capacity of approximately 560 psi, which is sufficient to overcome the creep stresses without any contribution from shear friction.

All of the concrete columns were field cured at an ambient temperature of 55 degrees Fahrenheit and 40 percent humidity. While these values do not correspond to a standard ASTM test, they are representative of common construction conditions where structural members are field cured. The formwork was removed five days after the concrete columns were cast, and repairs were completed. Mixing and placement of concrete were performed in strict compliance with ACI 304R-00 under controlled conditions.

The observed compressive failure pattern at the columns at the same location is an indication that the test set-up was successful in determining the capacity of the concrete repairs. One of the most important decisions related to the specimen preparation was the extension of the vertical bars and how to make the test most meaningful. As shown in **Figure 1-1**, vertical rebar at one end of the column was not completely extended to the surface of the column, and 2.75" to 3" of the concrete cover was provided from the top of the rebar to the concrete surface. As explained in Chapter 1, the continuation of the vertical bars into the footing is essential in vertical load transfer and obtaining the axial load capacity of the rebar within a column. In the absence of vertical bars continuing into a footing, axial load capacity is limited to the bearing capacity of the concrete (ACI 318-14). Hence, to be able to limit the failure mechanism of the 1/3 scale columns without a foundation to a purely compressive failure (in bearing), rebar was placed 2.75" below the column surface.

Observed failure modes with the compression cone surrounding the rebar indicate that at every column, the column capacity was equal to the bearing capacity of the concrete as predicted by ACI 318 (*2014*). While an argument could have been made by extending the rebar to the column surfaces, to allow both surfaces of the column to have the same conditions, it would have made the test less realistic. If both ends of the samples could have been completely restrained with rigid connections (i.e., mimicking the condition where dowels would be entering into a foundation at the base as in the case of a continuous column), it would have made sense to extend the rebar to the column surfaces at both ends; however, the utilized axial load testing equipment did not have such capabilities to create rigid ends.

Another reason behind the chosen failure pattern is for the determination of the axial deformation value for the concrete. As the failure was purely compressive, obtained deformation and strain values give an idea about the compressive strain in repaired and unrepaired columns without the participation of the rebar. Control samples and deep repairs both had an average strain of 0.008, while the shallow repairs had an average strain of 0.009, which is a very small difference.

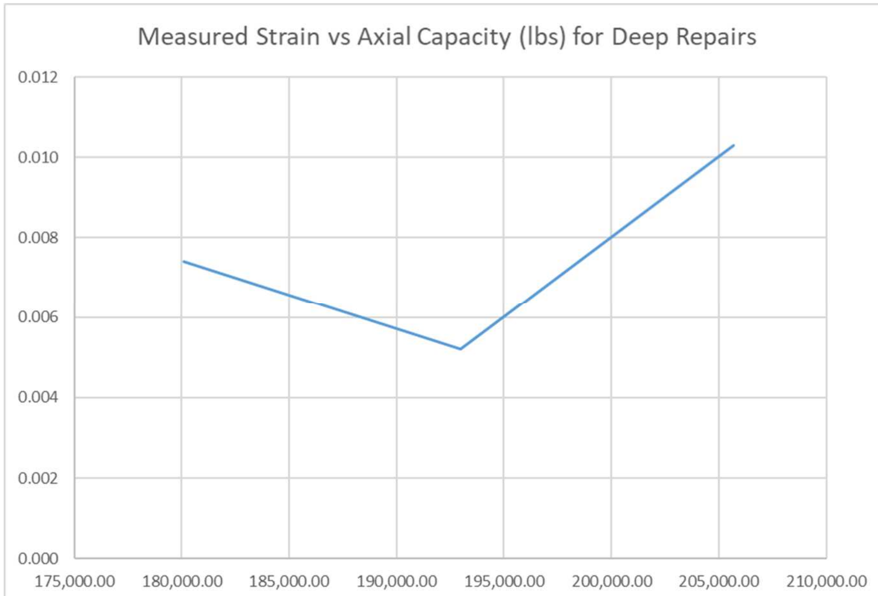


Figure 4-3-Axial Capacity vs Measured Strain for Deep Repairs

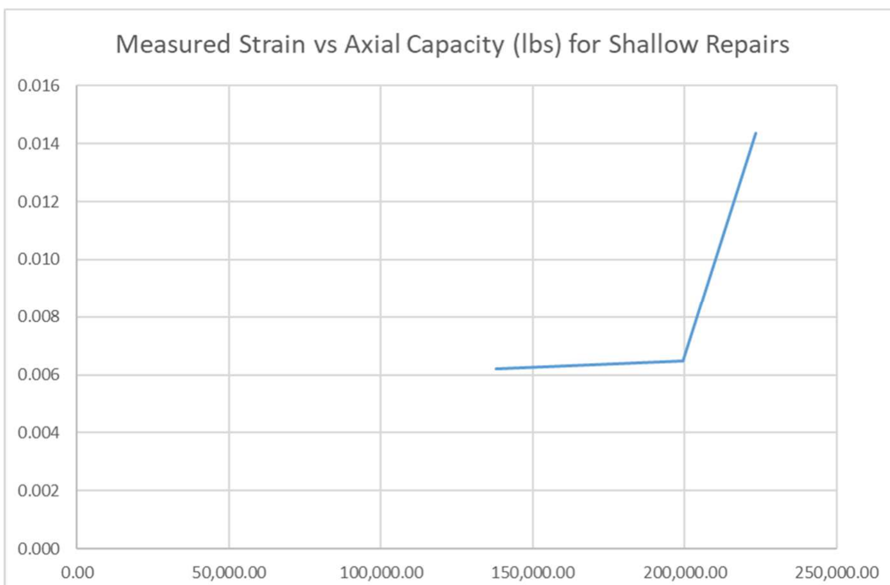


Figure 4-4-Axial Capacity vs Measured Strain for Shallow Repairs

Figures 4-3 and 4-4 above depict the variation of strain at failure against the axial capacity. Observed non-linear relationship is most likely due to the material variations and sensitivity of the equipment. Observed maximum deformation values and average values are very close to for repaired and unrepaired columns as seen in **Table 4** below.

Sample ID	Initial Length (in)	Max. Disp. (in)	Strain	Dia. (in)	Area (in^2)
Ave. for DR	36.15	0.276	0.008	8.03	50.66
Ave for SR	36.56	0.329	0.009	7.91	49.08
Ave for Control	36.42	0.284	0.008	8.00	50.24

Table 4-Average Deformation Values

As explained, this research intended to replicate the actual methods used in the field and perform the repairs in accordance with the recommendations of the American Concrete Institute's various guides, particularly 546 (*2014*). Samples were poured and repaired under controlled conditions, and concrete demolition was performed with a handheld electric chipping hammer with a chisel tip (weighing 2.4 lbs) to reduce cracking and eliminate fissures. Repair areas were visually inspected after the demolition was completed to ensure that all cracked, chipped, or fractured concrete surfaces were completely removed down to sound concrete, and the surfaces were roughened to ¼ inch magnitude as per ACI 318 (*2014*). During the entire testing process, every effort was made to ensure that the repair materials were completely bonded to the substrates, and cracks were avoided. 7-days prior to testing, top and bottom surfaces of the columns were scored (to ¼" magnitude by using a rotary wheel to create a rough surface), a latex-based concrete bonding agent was applied to the scored (and cleaned surfaces) along with an 8,000 psi gypsum cement leveling material (hydrocal) to ensure that that t the top and bottom surfaces are completely straight. The thickness of the hydrocal varied between 1/8-in to ¼-in (depending on the surface levelness). After the hydrocal application, it was ensured that the material was completely bonded to the concrete by tapping the top surfaces with the aid of a small steel rod to check for delamination. It was ensured that hydrocal completely adhered to the concrete.

Chapter 5: Conclusions and Recommendations for Future Work

A comparison of the results indicates that shallow and deep repairs performed at reinforced concrete columns are effective in restoring the original column capacities and potentially cause an unintended increase. As noted, in the case of deep repairs, a slight increase in the capacity, possibly due to the use of higher-strength materials, was seen.

Columns remained unloaded during repairs, and the only time load was applied was during the load testing. Thus, load redistribution within the column sections occurred soon after the load application, which allowed the repaired and unrepaired sections of the columns to participate. In the case of columns being repaired under loading (at least partial dead loading), this participation is expected to occur after live loads are re-applied. Especially if the live loads are considerable, it can be concluded from the performed testing that load redistribution would occur. Considering the minimal shrinkage within the patch material due to the use of shrinkage compensating concrete, the engagement of the repair material is not a concern, and the effects of creep /shrinkage are minimized. If shrinkage compensating materials are not used, and drying shrinkage is not controlled, there will likely be deformation in the column section before the repaired sections can engage, which will cause the column to sustain an additional internal localized moment (which may cause cracking rather than failure).

Another important conclusion of this research is that the repairs did not start to spall or delaminate before the failure of the columns. This observation is indicative that the load transfer is accomplished by shear friction between the original concrete and repair materials. Surface preparation methods utilized in this research are found to be appropriate.

The performance of the columns under dynamic loading conditions (while sustaining shear and flexure along with axial loading) requires further research into the topic. Concrete repairs are commonly performed in seismic zones throughout the world, and it is unknown whether the dynamic loading patterns caused by the earthquakes would cause the patch materials to delaminate or any special precautions are needed (such as wrapping column repairs with FRPs or steel jacketing) to ensure adequate performance during seismic events. Such concerns were raised for epoxy doweling materials used to anchor rebar and bolts into concrete, which led to further research and certification of such materials under dynamic conditions. The performance of repaired buildings during the earthquakes will likely be questioned in the future, and provisions regarding such repairs will need to be implemented to protect their structural integrity.

As for the axial load capacity under static loading conditions, further testing can be performed by utilizing full-scale testing for determining the effects of dowels into the foundations and whether the column can be made stronger by utilizing stronger concrete mixes in deep repair applications; however, the results of this research are conclusive in terms of the adequacy of chip and patch methods used to repair concrete columns.

Appendix 1 – Repair Mortar Used

SELF-CONSOLIDATING CONCRETE REPAIR MORTAR

EUCLID CHEMICAL

DESCRIPTION

EUCOREPAIR SCC is a versatile, one component, self-consolidating repair mortar that is shrinkage-compensated, polymer and microfiber modified, and contains an integral corrosion inhibitor. It is designed for horizontal and formed vertical/overhead structural repairs in applications from 1 inch (2.5 cm) to full depth.

PRIMARY APPLICATIONS

- Parking decks
- Joint repairs
- Balconies
- Equipment bases
- Pavements
- Beams
- Vertical and overhead formed repairs

FEATURES/BENEFITS

- Shrinkage compensation and reduction to minimize cracking
- Pre-mixed with pea gravel, ready-to-use
- Low permeability with excellent freeze-thaw resistance
- Polymer and microfiber modified
- Interior or exterior use
- Contains an integral corrosion inhibitor
- Long working time

TECHNICAL INFORMATION

The following are typical values obtained under laboratory conditions. Expect reasonable variation under field conditions.

Compressive Strength ASTM C39, 3 x 6" cylinder @ 0.5 gal/50 lb. bag.
Age
1 day..............2,750 psi (19 MPa)
3 days..............4,100 psi (28 MPa)
7 days..............4,800 psi (33 MPa)
28 days..............5,600 psi (39 MPa)

Freeze/Thaw Resistance ASTM C666 Procedure A
300 cycles..............>98% relative dynamic modulus

Sulfate Resistance ASTM C1012
6 months..............+0.005%

Surface Resistivity @ 28 days........31,200 ohm-cm

Flexural Strength ASTM C78
1 day..............450 psi (3 MPa)
7 days..............800 psi (6 MPa)
28 days..............900 psi (6 MPa)

Rapid Chloride Permeability ASTM C1202
28 days..............1,800 coulombs

Length Change ASTM C157, 50% RH @ 23°C (73°F)
(3" x 3" x 11" specimens were removed from molds @ 24 hours)
28 day shrinkage..............<0.050%

Slump Flow ASTM C1611
Initial..............24 - 33 inches (66 cm)
30 minutes..............24 - 33 inches (66 cm)

J-Ring Slump Flow ASTM C1621

25 inches (63.5 cm)
Passing Ability: 0.75 inch (1.9 cm) no visible blocking

Set Time ASTM C403
Intial..............approx. 9 hrs

Fresh Wet Density ASTM C138
144.4 lb/ft^3 (2313.1 kg/m^3)

Slant Shear Bond Strength ASTM C882

7 days..............2,400 psi (17 MPa)
28 days..............2,560 psi (18 MPa)

Crack Resistance ASTM C1581
Net Time Until Cracking: 15.5 days
Stress Rate: 16 psi/day
Potential for Cracking: Moderate-Low

PACKAGING/YIELD

EUCOREPAIR SCC is packaged in 50 lb (22.7 kg) bags.

Yield: Approximately 0.375 ft^3 (0.01 m^3) per bag when mixed with 0.5 gal (4 pints) of water. Bulk bags suitable for mixing in ready-mix trucks are also available.

Appendix 1-Repair Mortar (Courtesy of Euclid Chemicals)

References

1. ACI Committee 318. (Eds). (2014). *Building Code Requirements for Structural Concrete (ACI 318-14)*, Farmington Hills, MI: American Concrete Institute.

2. ACI Committee 201. (Eds). (2008). *Guide to Durable Concrete (ACI 201.2R-08)*, Farmington Hills, MI: American Concrete Institute.

3. ACI Committee 546. (Eds). (2014). *Guide to Concrete Repair (ACI 546R-14)*, Farmington Hills, MI: American Concrete Institute.

4. ACI Committee 224 (Eds). (2012). *Causes, Evaluation, and Repair of Cracks in Concrete Structures* (ACI 224.1R-07), Farmington Hills, MI: American Concrete Institute.

5. ACI Committee 364 (Eds). (2010). *Importance of Modulus of Elasticity in Surface Repair Materials (ACI 364.5T-10)*, Farmington Hills, MI: American Concrete Institute.

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