MID-AMERICA TRANSPORTATION CENTER

Report # MATC-KSU: 110

Final Report

Assessing the Damage Potential in Pretensioned Bridges, Caused by Increased Truck Loads Due to Freight Movements (Phase I)

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2010

A Cooperative Research Project sponsored by the U.S. Department of Transportation Research and Innovative Technology Administration

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A Report on Research Sponsored By

Mid-America Transportation Center
University of Nebraska-Lincoln

May 2010

1. Report No. 25-1121-0001-110	2.	3. Recipient's Accession No.
4. Title and Subtitle Assessing the Damage Potential in Pretensioned Bridges, Caused by Increased Truck Loads Due to Freight Movements (Phase 1)		5. Report Date May 2010
		6.
7. Author(s) Robert J. Peterman and Steven Hammerschmidt		8. Performing Organization Report No. 25-1121-0001-110
9. Performing Organization Name and Address		10. Project/Task/Work Unit No.
		11. Contract © or Grant (G) No.
12. Sponsoring Organization Name and Address Mid-America Transportation Center U.S. Department of Transportation Region VII University Transportation Center University of Nebraska-Lincoln 2200 Vine Street P.O. Box 830851 Lincoln, Nebraska 68583-0851		13. Type of Report and Period Covered
		14. Sponsoring Agency Code MATC TRB RiP No. 17132
15. Supplementary Notes		

16. Abstract (Limit: 200 words)

When evaluating the condition of existing bridges determining in situ stresses in the members provides valuable information about the condition of the structures. In this study, a method of surface strain relief was investigated whereby the change in strain at the surface of concrete members is used to determine the in situ stress. The method involved mounting a linear electrical-resistance strain gage along the axis of maximum stress, coring around the gage, and then relating the change in strain to the corresponding stress in the member.
Members were fabricated and varying stresses were applied in order to determine the accuracy of the method. Results were then compared to the global stresses and to the theoretical local stresses predicted by two different finite element models. In order to improve the accuracy of the surface-strain relief method, a procedure was introduced whereby the core was fractured along its base and subsequently removed from the member. This served to eliminate possible shear stresses between the core and surrounding member, allowing for the full release of strains.

17. Document Analysis/Descriptors		18. Availability Statement	
19. Security Class (this report)	20. Security Class (this page)	21. No. of Pages 35	22. Price

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Acknowledgements

The authors would like to thank the Mid-American Transportation Center (MATC) for funding this project, and Dr. Mustaque Hossain, MATC Associate Director at Kansas State University, for his encouragement and guidance throughout the project. Finally, sincere thanks are extended to Mr. Jake Perkins and Mr. Shahin Mayyeriamiri who worked diligently on the first portion of this research project.

Abstract

When evaluating the condition of existing bridges determining in situ stresses in the members provides valuable information about the condition of the structures. In this study, a method of surface strain relief was investigated whereby the change in strain at the surface of concrete members is used to determine the in situ stress. The method involved mounting a linear electrical-resistance strain gage along the axis of maximum stress, coring around the gage, and then relating the change in strain to the corresponding stress in the member.

Members were fabricated and varying stresses were applied in order to determine the accuracy of the method. Results were then compared to the global stresses and to the theoretical local stresses predicted by two different finite element models. In order to improve the accuracy of the surface-strain relief method, a procedure was introduced whereby the core was fractured along its base and subsequently removed from the member. This served to eliminate possible shear stresses between the core and surrounding member, allowing for the full release of strains.

Executive Summary

Phase I of the project was "Development of a Procedure to Determine the Internal Stresses in Concrete Bridge Members." A method of surface strain relief by dry-coring was established and tested in the laboratory with varying members and applied stresses. Different methods were researched and considered in the development of the surface strain relief method. These methods included the ASTM hole-drilling strain-gage method, the concrete stress-relief core method, and the concrete core trepanning method.

Linear electrical-resistance strain gages, having a 25.4 mm gage length, were attached to the surface of a member along the axis of maximum stress. After the gages were mounted and the output readings zeroed, a 63.5 mm outer-diameter diamond core bit was used to core around the gage. Through experimental and finite element model analyses a core depth of 25.4 mm was determined. After coring, the change in strain was measured and related to the internal stress.

The initial laboratory member was a 127 x 127 mm concrete beam with a total length of one meter. This member was loaded using an MTS servo-controlled test frame with loads of 271.5 kN and, afterwards, 273.3 kN. The first gage tested resulted in a discrepancy in the applied stress to the measured stress of 4.5 MPa. The second gage cored had a lower applied load and yielded much more accurate results with an error of only 0.1 MPa. The error in the first gage was believed to be due to the load being applied eccentrically. This resulted in the applied load producing not only a compressive force but also a moment—resulting in the discrepancy between the applied and measured stresses.

To eliminate the error from the load not being applied concentrically, additional test members were created with a single strand placed along the centroid of the member that was post-tensioned prior to coring around the gages. These specimens had a 102 x 102 mm square cross section and a total length of 4.90 m. Strain gages were then mounted at the fifth points along the top face of the member. Four test members with this cross section were cast and tested with varying stresses applied to each. With the first two members, the gages were mounted, cored around, and the change in strain recorded with the cores intact. For the last two members, after the change in strain was recorded, the cores were fractured along the base and then removed from the member, providing a full release of the stress. When the core was fractured along the base a higher level of accuracy was achieved each time with an average error less than 2%.

The research team concluded that the results showed a high level of accuracy and recommend continuation to Phase II. The laboratory tests showed that the 25.4 mm stain gage worked well when applied to a mortar mix but needed to be evaluated with other aggregate sizes. Phase II will utilize the surface strain relief method on pre-tensioned concrete members with the prestressing force being determined by using a crack-opening procedure. If Phase II yields successful results, testing on actual bridge members is planned (Phase III).

Chapter 1 Introduction

When determining the condition of existing concrete bridges and their elements, the ability to accurately determine the in situ member stresses would enable the engineer to make proper assessments. Unfortunately, the in situ stresses cannot be readily determined in most structures because information about the load distribution and restraint of time-dependent deformations is unknown.

Live-load testing is often employed in order to quantify the load-distribution aspects of a structure. In typical live-load tests strain gages are mounted on the surface of members, external loads are applied, and the increase in strain in the members is determined. While these techniques are routinely used to determine the change in stress due to applied loads, they cannot evaluate the absolute level of stress in the bridge as the initial stress condition is unknown.

Several different methods have been previously proposed to determine the initial stresses in structural members. A method typically used for determining the in situ stresses in metals is the ASTM hole-drilling strain-gage method (ASTM E837-08), which is used to determine residual stresses near the surface of a material. This method requires that a rosette strain gage is attached to the member surface, and then a small hole (diameter 1-2 mm) is drilled in the center of the rosette strain gage and the relief stresses are captured. However, this method cannot be readily adapted for use with concrete and, as such, the ASTM method is applicable to homogeneous isotropic linear-elastic materials.

Other methods that have been proposed for testing concrete members include the concrete stress relief core method and the steel stress relief hole technique (Kesavan

2005). The stress relief hole technique involves drilling a 1.6 mm diameter hole until reaching the surface of the rebar or prestress steel, and measuring the relieved strains close to the hole. This method exposes the reinforcement and is only applicable to small-diameter prestressing wires. The concrete stress relief core method involves mounting four strain gages around the location of the core hole along the principal strain planes, coring to a depth equal to the diameter, and measuring the relaxation around the boundary of the core. This method measures only a partial strain release and may produce large errors due to small measurable quantities.

A method very similar to the one used in this study is the concrete core trepanning method. This method uses a 30 mm strain gage with a 50 mm diameter diamond core bit. The single-strain gage is aligned in the direction of the maximum stress. The drilling takes place in 10 mm increments with water cooling the bit and strain data are recorded until the depth equals the diameter of the hole. Extra precautions in waterproofing of gages and special lead wire connections are needed to minimize errors during measurements. The advantages of this method include (1) the full release of internal strains and (2) the need to only collect data from one strain gage. The released strain is of opposite polarity compared to the in situ strain, so determining the in situ stress only requires a change in the sign and multiplying the strain by the modulus of elasticity (Kesavan 2005).

Phase I was used to evaluate a potential method of determining the existing stresses in a concrete member in the field. The project served to expand the research previously conducted by Kesavan. Through laboratory experiments and finite element analyses the optimum depth of coring was determined. At the outset of this study it was desirable to establish a single optimal depth of coring to be used, rather than to sequentially core in 10 mm increments as done in the core trepanning method. Concrete specimens were cast and known axial stresses applied then strain gages were installed and cored around in order to measure the “re-bounding” strain, and relate the strain to the known stress.

The method investigated in this study utilized strain gages that were mounted to the surface of the concrete. Next, a hand-held drill with a diamond core bit was used to core around the strain gage without the use of water, and the “re-bounding” of strain was related to the in situ stress. The elimination of wet-coring process was believed necessary for the following two reasons:

1. The introduction of water to the concrete surface could result in significant errors due to localized swelling of the concrete as the existing shrinkage strains are reversed.
2. The use of a wet core-bit and large core-drill adds an undesirable complication to the ability of this method to be used in the field. Note that the core trepanning method was developed for use in the laboratory.

Chapter 2 Instrumentation Required for Surface Strain Relief Method

At the beginning of Phase I a dry method of surface strain relief was established. In order to define the process, the size and availability of strain gages and diamond core bits were researched. From the gathered information, recommended mounting procedures for strain gages were considered and a method was determined to core around the gage. Finally, a method for recording values and comparing results to theoretical calculations was determined.

2.1 Determination of Strain Gage

The ASTM hole-drilling strain-gage method uses a rosette strain gage, but for the purpose of this project, the orientation of maximum stress was assumed to be known. With the orientation of the maximum stress known, a linear strain gage could then be used. In Phase I, a mortar mix was used in order to minimize the length of the strain gage required.

A general rule of thumb for concrete strain gage work is to make sure that the strain-gage grid is about five-times larger than the largest size of aggregate (Vishay 2005). In the initial tests the largest aggregate had a maximum size of 5 mm. Since a wide array of strain gages exist, a gage with a durable backing and applicable strain range was selected. The gages used in Phase I were Vishay Micro measurements type EA-06-10CBE-120. These gages had a grid length of 25.4 mm and a grid width of 6.35 mm.

2.2 Determination of Core Bit

Once the gage was selected, an appropriate-size diamond concrete core bit was chosen due to its durability and ability to core around the gage without the use of water. Note, the coring was completed without any impact, such as those created by a “hammer drill," which could damage the core or induce additional stress. The overall dimensions of the strain gage were 34.5 mm by 8.4 mm. Additional space was needed to solder the lead wires to the gage and mount a connection device to allow for quick connection and disconnection between the strain gage and instrumentation.

In consideration of these parameters, a core-bit with an outside diameter of 63.5 mm was selected. The inside diameter of this bit was approximately 57.2 mm, leaving about 11 mm of clearance on each side of the gage to minimize disturbance of the gage during the coring process. The concrete core trepanning method used a 30 mm strain gage and a 50 mm diamond core bit (Kesavan 2005).

2.3 Determination of Depth of Core

To determine the depth of the core, laboratory and finite element models were created. The concrete core trepanning method states that the depth of the core should be equal to the diameter of the core bit (Kesavan 2005). Using a depth of the core equal to the diameter of the core bit presented the possibility of damaging any steel reinforcement that is placed near the concrete surface with minimum concrete.

Through laboratory experiments the researchers found that a coring depth of 25.4 mm yielded acceptable results. This depth was found to adequately release the stress and to minimize internal stresses caused by the concrete core remaining attached to the member. The depth of 25.4 mm was verified by coring a specimen with a square cross section to varying depths, and then comparing the results to finite element models as explained below.

A finite element model was created using SolidWorks 2009 and it showed the varying depths of cores and the stresses and strains related to each. Two different square cross sections were created to simplify the creation of and to reduce the size of the model. The model contained a near constant stress over the cross section. This was achieved by applying restraint on each end to prevent movement away from the longitudinal axis.

Additional restraint was applied to the midpoint to prevent movement along the longitudinal axis, and the load was applied to each end—simulating a prestressing stress of 13.79 MPa (2000 psi), as shown in figure 2.1. To reduce the localized influences of the applied loads and restraints, a simulated core was created in the center of the member, and the member was long enough to have a near constant stress throughout the area of interest. To simulate the core, the area was removed between the inside and outside diameter of the core bit, leaving a cylindrical hole. The core locations were centered in the model to reduce the possibility of the edge-effects causing variation in the stress on one side more than the other.

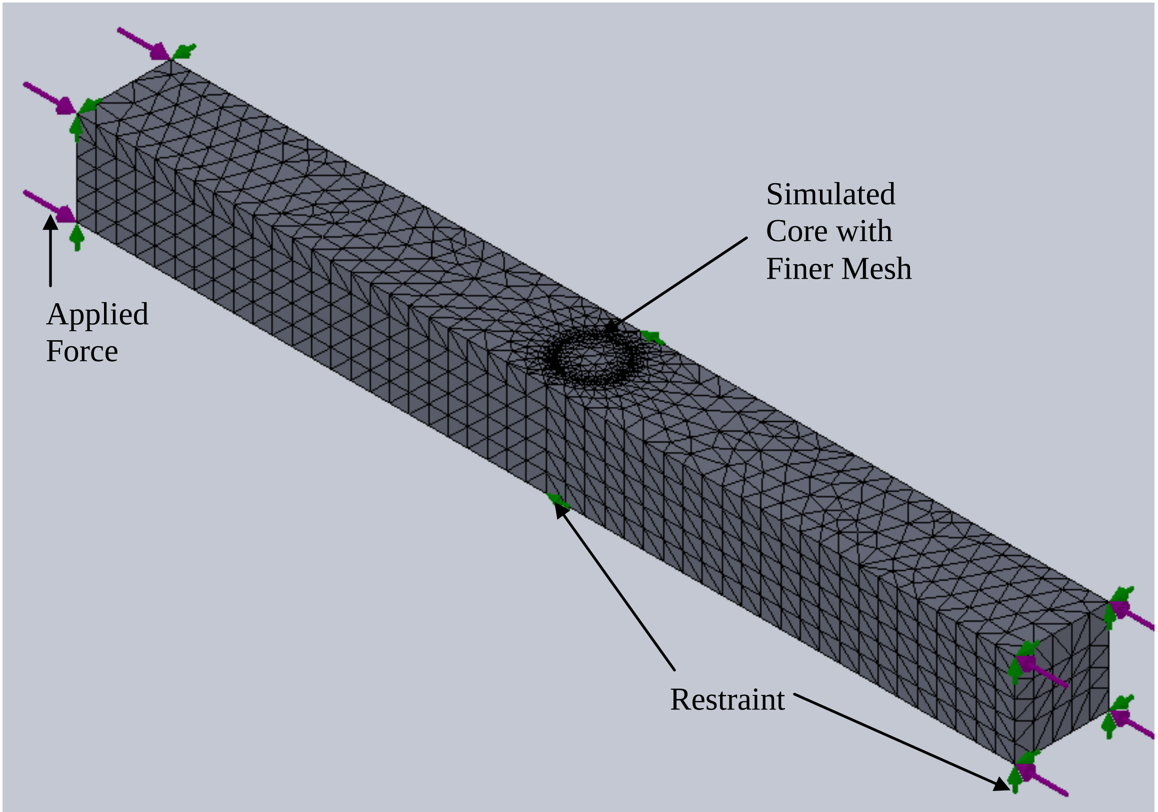

Figure 2.1 Finite element model mesh

A parametric analysis was conducted using two different sized members, varying the depth of the core, and plotting the stress values at the top surface and continuing down through the center of the core to a distance of about 12.7 mm below the bottom of the core. The first model represented a member similar to the post-tensioned test specimens evaluated in the laboratory. The model had a square cross section with sides of 102 mm and a total length of one meter. High-quality tetrahedral elements were used with a fine mesh applied with sides of 3 mm applied to the core and surrounding area. A high-quality tetrahedral element with a large size of 25.4 mm was applied to the remaining areas. To verify the model, stress was checked at a location away from the core to verify that it was equal to the average normal stress. A section was cut down the middle of the model showing the stresses at the center of the core as shown in figure 2.2 Stress values were obtained and plotted versus the depth as shown in figure 2.3. When the core depth starts to exceed 19 mm, a small tensile stress is created at the surface of the core. This surface stress is due to a shear stress that exists at the base of the core. As the depth of the core increases, the tensile stress at the surface increases slightly but not by a significant value.

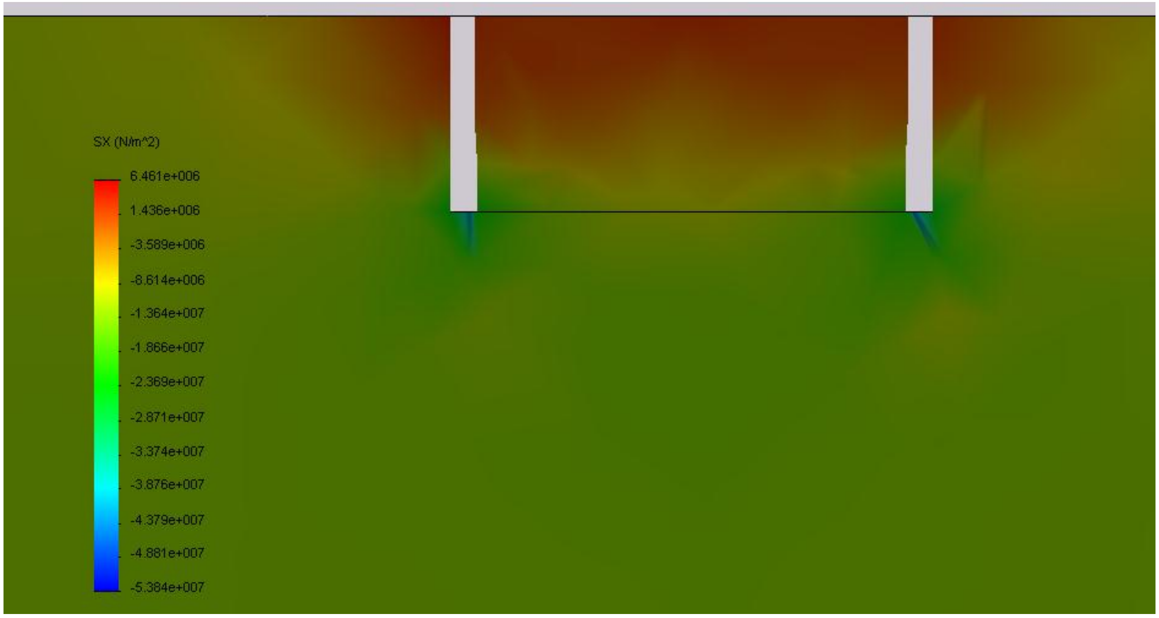

Figure 2.2 Stress distribution of the finite element model

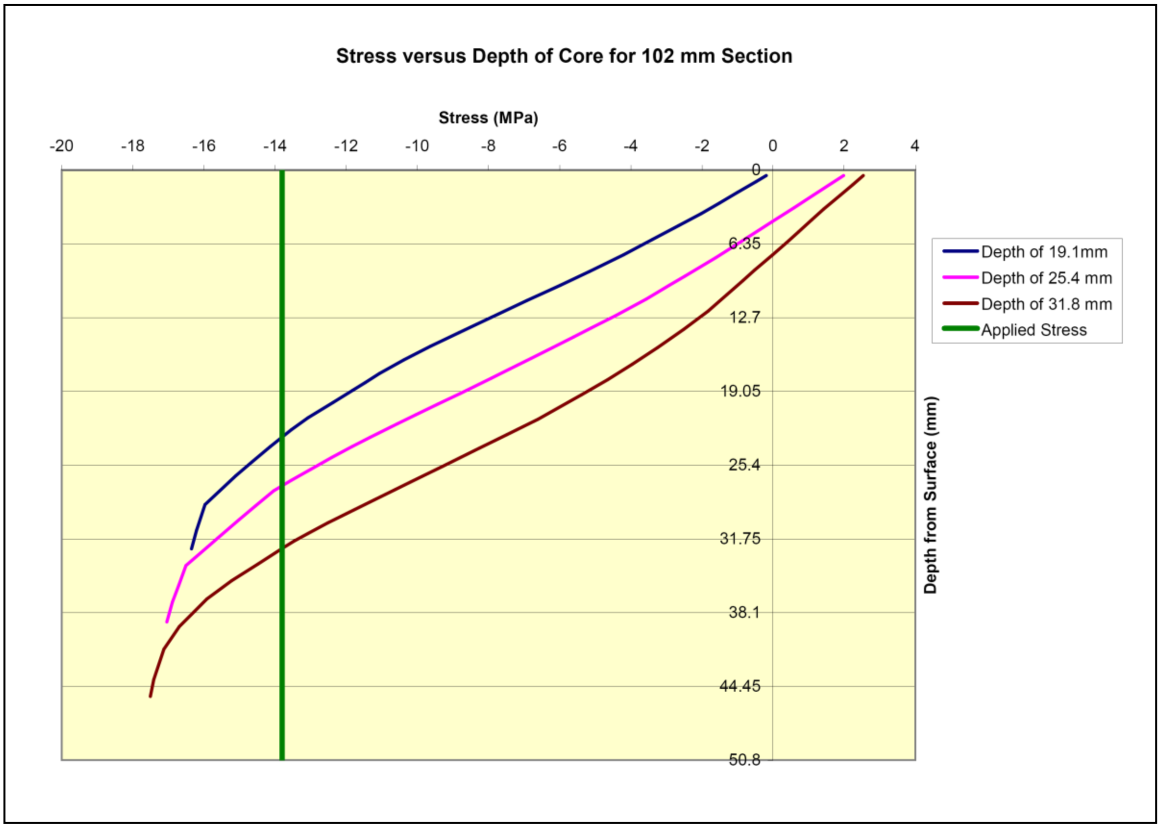

Figure 2.3 Stresses versus depth of core for 102 mm x 102 mm square cross section

A second model was created to verify that the positive stress at the surface was due to the shear at the base of the core, and not due to bending in the relatively slender member caused by the coring operation. This member had a larger square cross section with sides of 305 mm (approximately 3 times the size of the first model) and had a total length of one meter. The same restraints and mesh were applied along with a larger force to maintain the same constant stress. This second model yielded similar results. Thus, it was evident that the deeper depth equal to the core diameter, as recommended by Kesavan, would not be necessary.

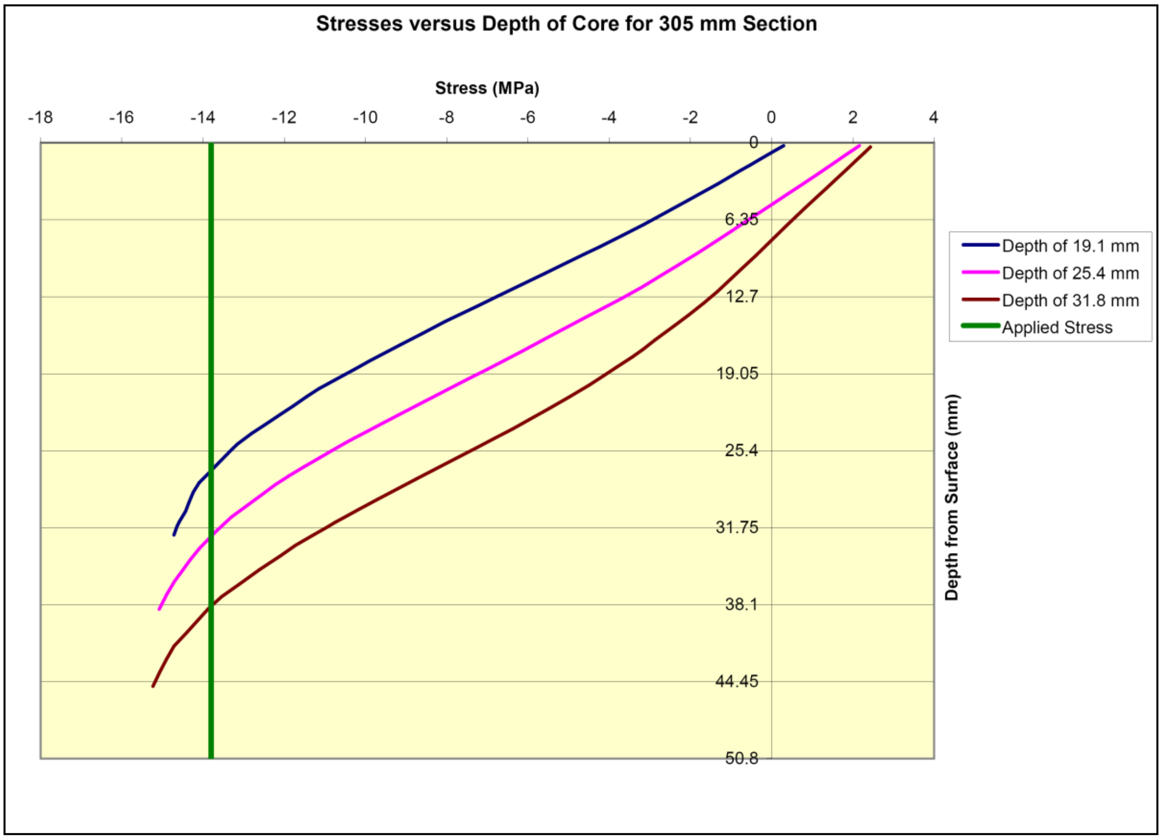

Figure 2.4 Stresses versus depth of core for 305 mm x 305 mm square cross section

In order to eliminate the transfer of strains between the member and the cored section the authors developed a method to extract the core out of the hole, thus relieving the internal shear stress. The depth of 25.4 mm was chosen to make sure that a core hole was drilled deep enough to relieve the stress in the core and to ensure that when the core was extracted, it would be deeper than the minimum depth of 19 mm. Note that when the core is fractured, the fractured plane is always less that the actual depth to which the core is drilled (refer to figure 5.5). Figures 2.3 and 2.4 each show a near-linear trend through the core depth. For each core depth, a positive (tension) stress is created at the surface due to the interaction between the core and the member, as the shearing stress along the bottom results in a stress at the surface. An increase in depth from 19.1 mm to 25.4 mm shows a jump of 1.6 MPa at the surface. At a depth of 25.4 mm to 31.8 mm the jump is very small. These models indicate that some calibration method or a method to remove the additional stress may be needed to improve the accuracy of the test results.

Chapter 3 Surface Strain Relief Method

The surface strain relief method included surface preparation, strain gage installation, coring of the concrete around the gage, recording strain changes, and validating results. With the strain gage selected and the diameter of the core bit set, the process of determining the in situ stress could begin.

3.1 Surface Preparation and Gage Installation

The first step was to prepare the surface prior to mounting the strain gage and, particularly, the contact surface had to be clean and free of laitance. To prepare the surface a sanding disk of 80-grit was used to remove the laitance, and the surface was cleaned to remove any dust from the sanding. Using a ball point pen, layout marks were applied to the specimen to properly align the strain gage. The layout marks specified the axis of maximum stress and a line perpendicular to the maximum stress at the center of the location of interest.

Once this was completed, an epoxy was applied to fill in any voids in the concrete and create a level surface to mount the strain gage. M-Bond AE-10 was used as the epoxy and was obtained from Vishay Micro measurements. A thin layer of epoxy was applied to the surface, filling all voids and creating a level surface on which to mount the gage. The epoxy was allowed to cure a minimum of 12 hours before proceeding.

Next, the epoxy was dry-abraded using 220 to 320 grit sand paper. Then, it was wet-abraded using Conditioner A (supplied by Vishay) and 400 grit sand paper, and then the residue was removed with a gauze sponge. Once the area was sanded, it was cleaned and neutralized using Neutralizer 5A. The strain gage was then aligned and adhered using M-Bond AE-10 epoxy. A clamping force of 34.5 to 137.9 kN/m^2 was applied using steel weights until the epoxy cured. Next, the lead wires were soldered onto the strain gage and the gage was covered with a proactive coating of microcrystalline wax. Using a voltmeter the resistance of the gage was checked to confirm that the gage was installed properly.

3.2 Calculating the Elastic Modulus

The elastic modulus of the concrete was determined according to ASTM C 469, the standard test method for static modulus of elasticity and Poisson's ratio of concrete in compression. The compressive strength of the concrete was determined according to ASTM C39. The modulus of elasticity was taken as the average value obtained from two cylinders.

ASTM C 469 specifies a loading rate of 241.3 ± 34.5 kN/m^2 per second. Using a 101.6 mm diameter cylinder, a loading rate of 890 Newtons per second was used. Each concrete cylinder was loaded to 40% of the average compressive strength. To determine the strain during testing, a compressometer was attached to the concrete cylinder with a gage length of 135.9 mm.

Three cycles of loading were applied to each cylinder, and the second and third cycles were used to determine the modulus of elasticity. The chord modulus of elasticity was used to determine the modulus to the nearest 344.74 MPa (ASTM C 469) as shown below.

$$E = \frac{(S_2 - S_1)}{(\varepsilon_2 - 0.000050)} \qquad (3.1)$$

where

E = chord modulus of elasticity (MPa)
S_2 = stress corresponding to 40% of ultimate load (kN/m^2)
S_1 = stress corresponding to a longitudinal strain of 0.00005 (kN/m^2)
ε_2 = longitudinal strain produced by stress S_2

Chapter 4 Initial Evaluation of Surface Strain Relief Method Using Un-Reinforced Members

Initial experimental tests were conducted with concrete specimens having an un-reinforced square cross section. Axial loads were applied to these specimens in order to produce stress levels that would be typical of that in prestressed concrete bridge girders. The specimens were placed in an MTS servo-controlled load frame, which enabled the evaluation of the procedure at different stress levels.

4.1 Cross Section and Material Properties

The concrete mix design used for the members was a “mortar mix” without a course aggregate. The mortar mix was chosen so that a smaller strain gage could be mounted on the surface, allowing a gage length of 25.4 mm to be five times larger than the maximum size of aggregate. The mix was designed using Type III cement with an average compressive strength of 37.9 MPa. The members had a square cross section with sides of 127 mm and a total length of one meter.

Wood forms were used to cast the members. When casting the members, concrete cylinders were made to test the compressive strength and to determine the modulus of elasticity of the mix. Multiple concrete cylinders were cast so the compressive strength and modulus of elasticity could be determined for each member prior to testing. The compressive strength was used to determine the maximum axial load to be placed on the member. The modulus of elasticity was used to relate the surface strain to the normal stress, which was then compared to the average normal compressive stress on the gross cross section.

4.2 Location of Strain Gages

To capture the surface strain, two strain gages were mounted along the centerline of the beam at the third points shown in figure 4.1. These gages were oriented parallel to the direction of applied load to capture the maximum strain rebound. The strain gages were mounted, as described previously, and protected with micro-crystalline wax for protection. A 6 mm hole was drilled through the beam at a distance of 25.4 mm from the gage. Figure 4.2 shows the 6 mm hole, allowing for the lead wires to pass through the center of the core.

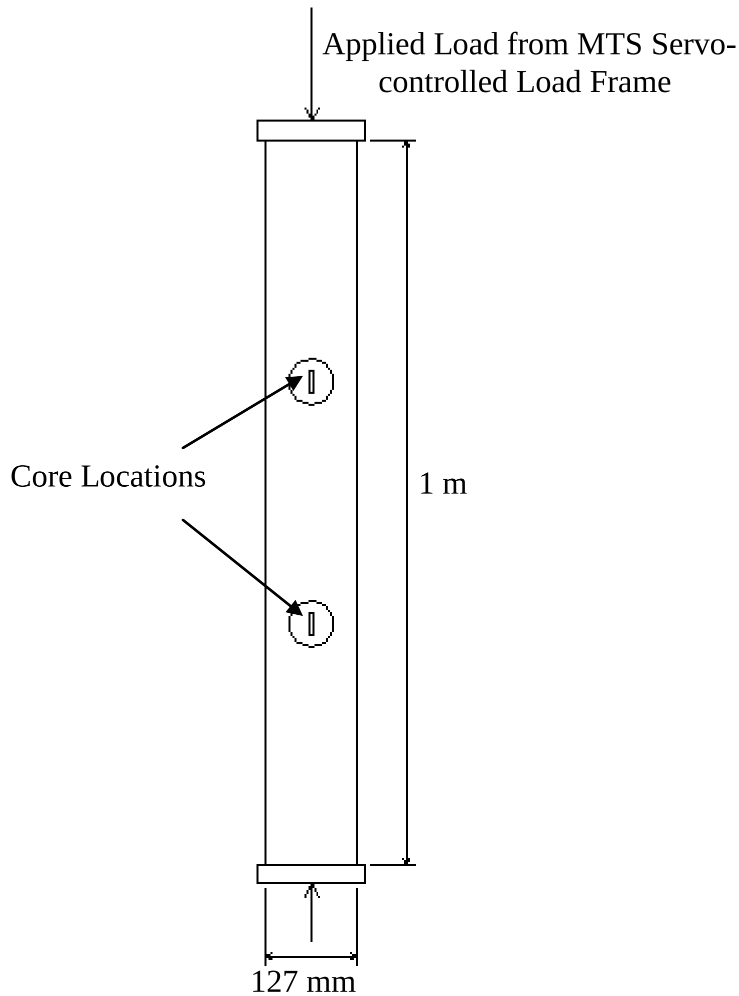

Figure 4.1: Schematic of initial test member

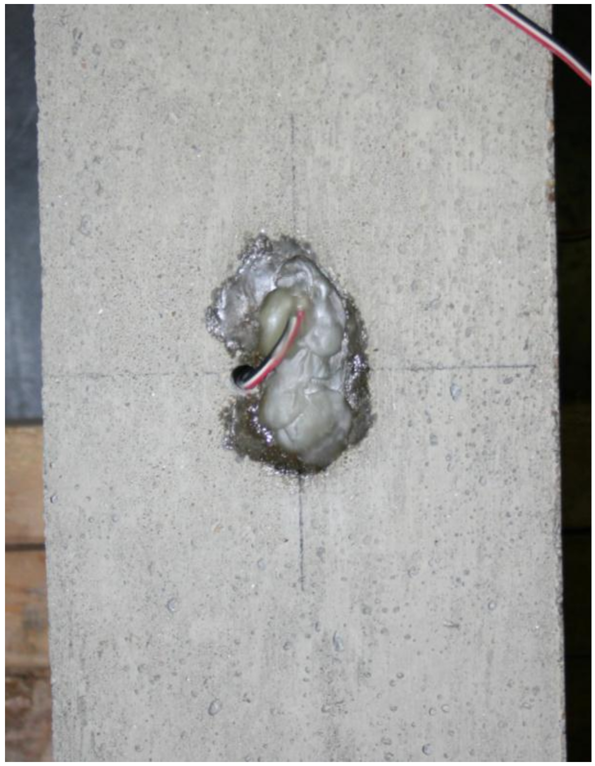

Figure 4.2 Strain gage mounted on member

4.3 Initial Testing

The specimen was placed in an MTS servo-controlled load frame and loaded to a stress level of approximately 60% of the compressive strength of the member. This targeted stress level was selected in order to simulate a stress level that would be representative of that in typical prestressed concrete members. Prior to loading, the lead wires from the strain gage were connected to a portable strain indicator (Vishay P-3500) to measure the strain values during the test. The gages were zeroed before applying the load. Once the load was applied, the strain reading was compared to the average compressive stress in the section using the corresponding modulus of elasticity for the mix.

$$\sigma = \varepsilon \cdot E \tag{4.1}$$

where

σ = the implied normal stress, ε = indicated strain, and E = chord modulus of elasticity

Once these values were recorded, a wooden guide was mounted around the beam as shown in figure 4.3. Locations of interest were then cored to varying depths, recording the strain at each depth. Water was not used to cool the diamond core bit. The use of water could possibly damage the strain gage, and the concrete absorbing water could influence the release of the strain from the core section. A vacuum was used to reduce the dust and keep the bit from binding, as shown in figure 4.4.

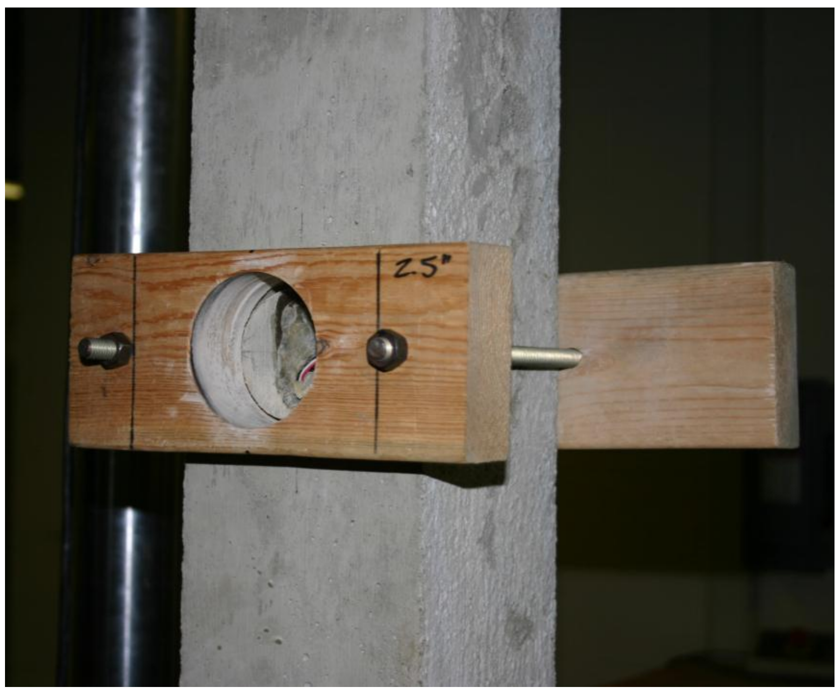

Figure 4.3 Wooden guide mounted on member

Figure 4.4 Coring around a strain gage

4.4 Initial Test Results

The average compressive strength of the concrete mortar mix, as determined from the concrete cylinders, was 35.2 MPa and the corresponding modulus of elasticity was determined to be 1.66 GPa. Table 4.1 shows the applied stress and the indicated stress from “re-bounding” strain values along with the corresponding percent error. For the first core, the gage had an initial strain reading of -733 micro strain. Immediately after coring, a strain value of -71 micro strain was recorded, with the value drifting to 5 micro strain after a short time.

For the second core, the load was reduced slightly (to adjust for the reduction in cross section from the first core) and this gage had an initial strain reading of -878 micro strain. This gage was cored to a depth of 38 mm with a corresponding strain value of -198 micro strain, drifting to a 6 micro strain after a short time. Figure 4.5 shows the core after drilling was complete.

Table 4.1 Initial test member stress values and percent error

Strain Gage	Applied Stress (MPa)	Implied "Re-bounding" Stress (MPa)	Percent Error
1	-16.8	-12.3	-27.1%
2	-14.7	-14.7	0.1%

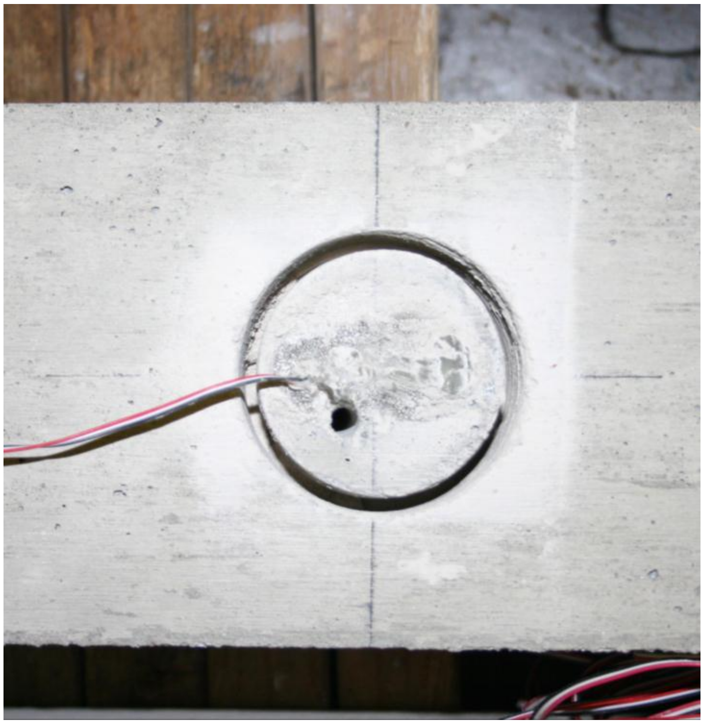

Figure 4.5 Photo showing member after core-drilling

A second beam was created using the same procedure, but large discrepancies were found relating the average normal stress to the related stress from the surface strain. The reason for these errors was believed to be caused by the ends of the beam not being perpendicular to the loading frame, thereby inducing a bending moment in addition to the axial force.

4.5 Conclusions from Initial Tests

The initial test member showed that the surface strain relief method had the potential to provide accurate results. The large discrepancies found in the second test member were likely caused by the inability to concentrically load the member and produce a known stress level at the location of the core. Thus, a new member type was needed to ensure that a near-constant stress field could be achieved.

Chapter 5 Testing of Surface Strain Relief Method on Post-Tensioned Sections

Because of the difficulties encountered in loading the unreinforced concrete members in a concentric manner, a second type of member was fabricated and tested. These members had a smaller cross section and were concentrically post-tensioned. The post-tensioning allowed a single strand to be placed along the centroid of the section. This strand reduced the possibility for load eccentricities and was more representative of actual prestressed concrete members.

5.1 Cross Section and Material Properties

The post-tensioned members had a square cross section of 102 mm by 102 mm and a total length of 4.90 m. These members had a centrally located single 12.7 mm diameter strand, which was tensioned by jacking after the specimens had reached the desired compressive strength. These specimens were fabricated using a mortar mix as previously done for the un-reinforced specimens.

However, the mortar mix used in the post-tensioned specimens had a design compressive strength lower than that of the previous specimens. The lower compressive strength was used was to compensate for the lower maximum stress that could be applied to the smaller cross section. A single 12.7 mm 1860 MPa low-relaxation strand was initially tensioned to a maximum of 80% of the specified tensile strength.

This maximum jacking load was 147 kN, resulting in a near-constant axial stress in the specimen of 13.8 MPa. This stress was equal to 60% of the targeted compressive strength of the mortar mix. The mortar mix was designed to have a compressive strength of 24.1 to 27.8 MPa at five days. A Type III cement was used in the mortar mix.

Each of the post-tensioned members was fabricated using wood forms. A single 12.70 mm-diameter strand was positioned at the centroid of the cross section and preloaded to approximately 500 N in order to straighten the strand in the form. Once the strand was taut, a plastic de-bonding tube was used to prevent the concrete from bonding to the strand. To minimize the friction force between the strand and surrounding concrete, the strand was wrapped in two separate layers of debond tubing with duct tape loosely wrapped around the first layer.

After double-wrapping the taught strand, each specimen was cast using the design mortar mix. The mortar mix was batched in a 1 ¾ ft^3 capacity Lancaster counter-current pan mixer; mixing enough mortar to cast the specimen and four concrete test cylinders. The cylinders were later used to determine both the compressive strength and the modulus of elasticity of the mix. A concrete vibrator was used to ensure good consolidation throughout the member. Each member was then covered with plastic and allowed to cure for 24 hours. After the member was cured, the wooden forms were removed and the specimen was inverted so that the bottom (flat) surface was facing upwards in preparation for the strain gage installation.

5.2 Test Setup

Four strain gages were positioned at the one-fifth points along the centerline of the beam. The strain gages were positioned parallel to the length of the beam to capture the maximum strain as shown in figure 5.1. The surface of the beam was prepared and the gages were positioned in the manner described in Chapter 3. To load the beam, a steel plate was fastened to each end with gypsum cement (Hydrocal) to evenly distribute the load. On one end, a hydraulic jack was positioned over the strand, locking the strand in the jaws of the jack. On the other end, there was a load cell with a steel plate threaded into it, and on the back side of the load cell was a steel plate with a strand chuck, as shown in figure 5.2.

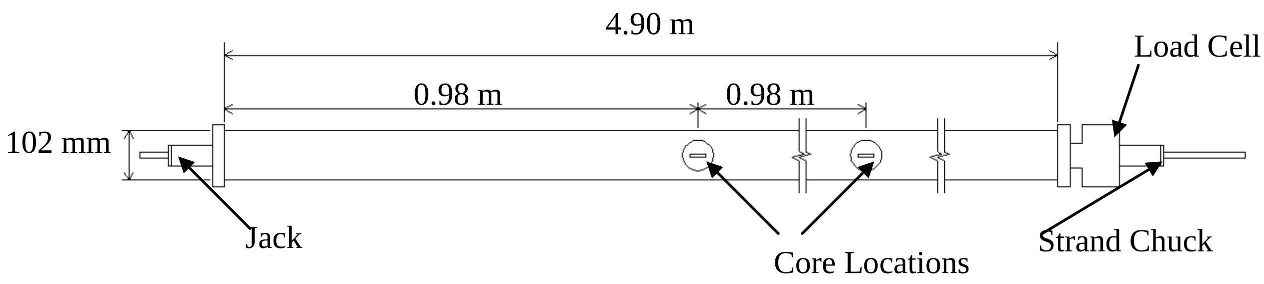

Figure 5.1 Schematic of initial test member

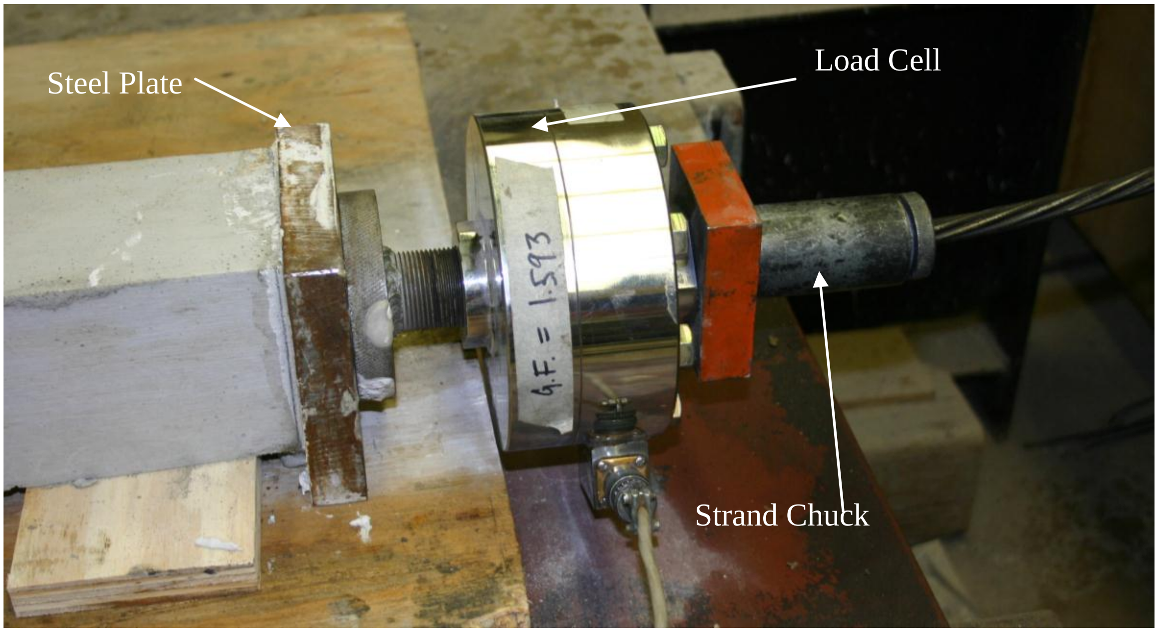

Figure 5.2 Load cell setup

Only short-term measurements were required of the strain gages so Vishay P-3500 instruments were used to connect, balance, and zero the gages. The strain gages were connected using a quarter-bridge configuration. Once the lead-wires were connected, the gage factor was set, the amp setting was set to zero, and the gage was balanced and set to an initial strain value of zero. After the strain gage had been zeroed, the strain was monitored for a few minutes to verify that the gage was not drifting or malfunctioning. The strand was then tensioned to the desired load. Different loads were applied to the members to measure and compare varying stress levels. Once the gage was determined to be working, the lead-wires were disconnected from the P-3500 at the connector and a coring guide was mounted around the strain gage.

5.3 Testing

A coring guide was used to steady the core bit when first rotating to prevent the bit from shifting on the specimen and damaging the strain gage or wires. The guide consisted of a hole drilled in a block of wood slightly larger than the outside diameter of the core bit. Attached to each side of the guide block was another perpendicular block of wood which served to center the core bit on the concrete specimen. Two clamps then secured the coring guide to the concrete specimen shown in figure 5.3. Before coring, a check was conducted to make sure the guide was centered on the strain gage, and the wires were secured out of the way to prevent any damage to them while coring. A hammer drill was used with the hammering feature turned off. A vacuum was used to reduce and remove the concrete dust from around the core bit. Once the core bit reached half the depth, the coring was stopped, the guide removed, and the dust removed from the hole. This was done to prevent the core bit from binding up and damaging, or possibly fracturing, the core. With the guide removed and the area cleared, the coring resumed to the desired depth of 25.4 mm. Using tape, the desired depth was marked on the core bit. Once the core bit reached the depth, the bit was removed from the hole, the dust vacuumed up, and the strain gage was then reconnected to the P-3500. Using a micrometer, the depth of the core hole was checked in multiple locations. The strain reading was then recorded along with the amount of time that had passed after the coring.

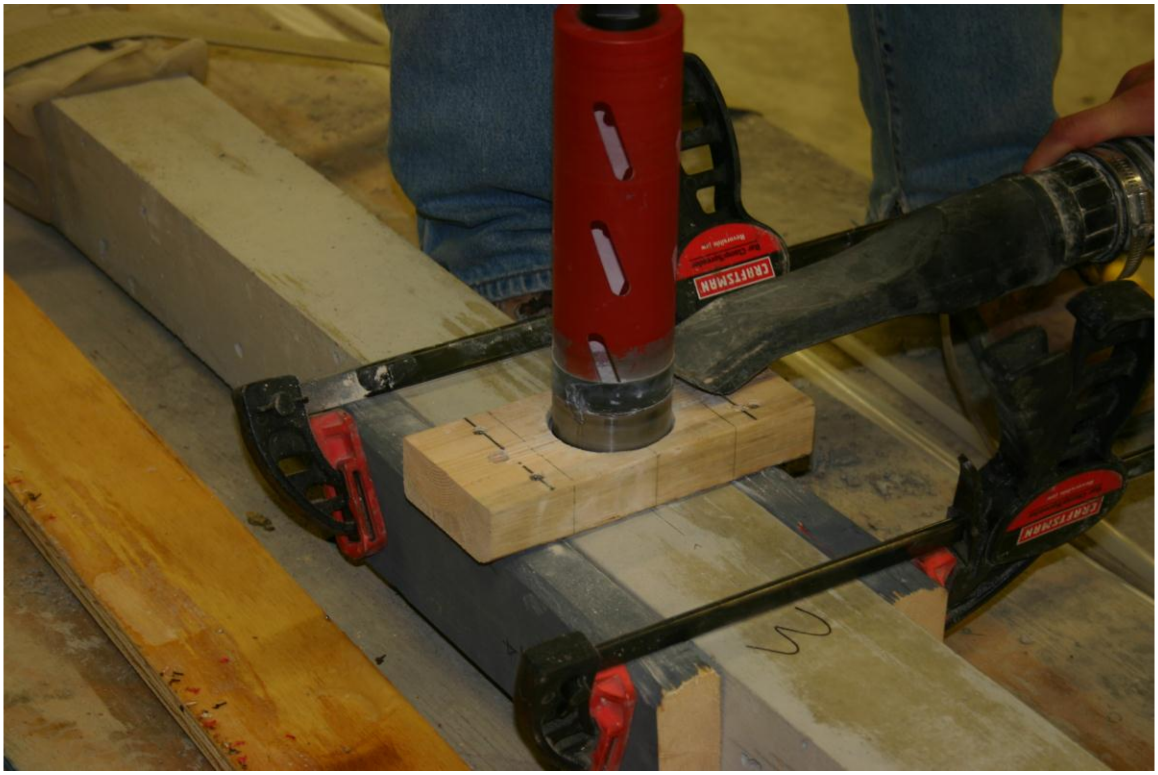

Figure 5.3 Coring the member

Through repeated trials a positive strain or a strain caused by a tension force was noticed at the surface of the core. To help reduce the amount of this strain, a method was devised to extract the core to relieve all internal strains. Using a large flat bar, that was inserted into the side of the core, and using steady pressure, the core was fractured along the plane of the bottom of the core as shown in figure 5.4. This caused a change in the strain value. The core was then removed, as shown in figure 5.5, and the value of strain was recorded along with the time, and the strain value was monitored over a period of four hours.

Figure 5.4 Fracturing the core center from the surrounding beam

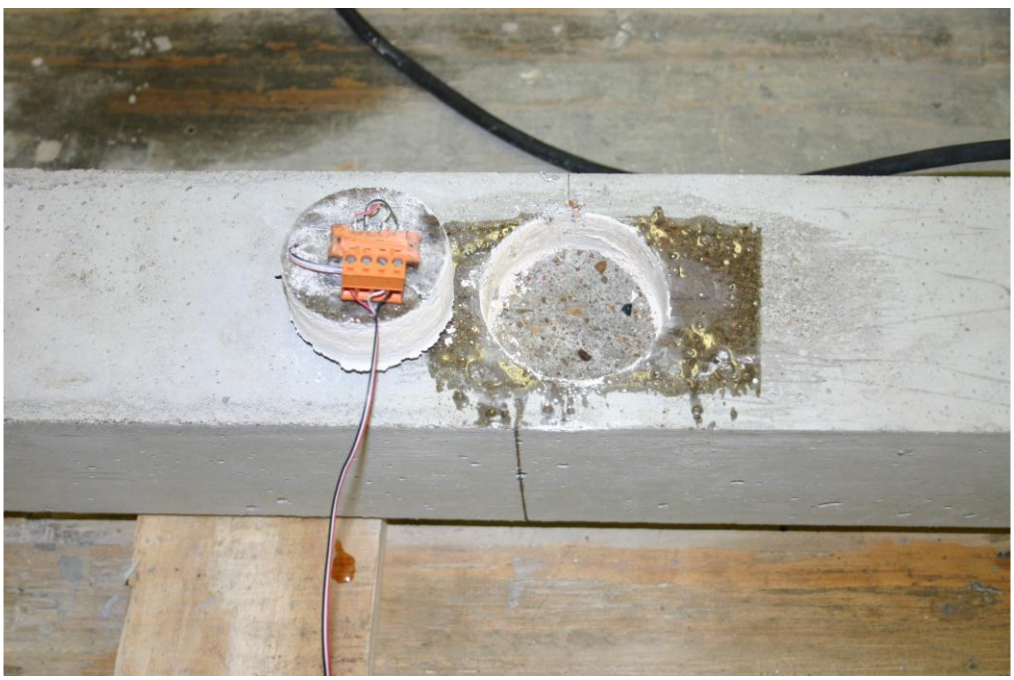

Figure 5.5 Core center removed from the member

5.4 Results from Post-Tensioned Tests

Four different post-tensioned beams were cast with similar mortar mixes and loaded to varying stress levels. The first two beams were cored, but the cores were not extracted, and a larger percent error was seen when relating the applied stress and the “re-bounding” stress. With the last two beams, each stain gage was cored, the value recorded, and the core fractured along the bottom. The new strain value was recorded, and this was related to the stress. The error when fracturing the core was reasonable. The load was applied on the member and held for four hours before any testing started. The load was held for four hours to minimize the effects of creep and shrinkage while the testing occurred. Each location was cored to a depth of 25.4 ± 1 mm and was verified around the core using a micrometer. Strain readings were taken immediately after the coring procedure was completed. The calculations were based on the initial strain measured before coring around the gage of interest, and the reading after the coring process was completed. The strain values over time are shown in Appendix A for each beam.

The first beam tested had an average compressive strength of 39.0 MPa and a modulus of elasticity of 2.38 GPa, found using ASTM C 469. This member was loaded to a compressive force of 142.4 N, creating an average normal stress of 13.8 MPa. The stresses are shown in table 5.1 and were derived so comparisons could be made between different beams without the influence of concrete material properties. The difference between the “applied” and “re-bounding” stresses are believed to be largely due to the shearing stress at the base of the core, which created a positive (tensile) stress at the concrete surface.

Table 5.1 Beam 1 Stress Values and Corresponding Errors

Strain Gage	Applied Stress (MPa)	"Re-bounding" of Stress with Core Attached to Member	% Error	Average Percent Error
1	-13.8	-15.1	-9.5%	-6.7%
2	-13.8	-14.4	-4.4%	
3	-13.8	-14.5	-4.9%	
4	-13.8	-15.0	-8.2%	

A second beam was tested with similar properties as the first one, but a lower compressive force was applied. Results are shown in table 5.2. The member had an average compressive strength of 39.6 MPa and a modulus of elasticity of 2.21 GPa. The member was loaded and a discrepancy was found between the load cell and the pump pressure, which was calibrated from the load cell. The conclusion was that friction had been created between the strand and the concrete, so a linear approximation was used to determine the load at each strain gage.

Table 5.2 Beam 2 Stress Values and Corresponding Errors

Strain Gage	Applied Stress (MPa)	"Re-bounding" of Stress with Core Attached to Member	% Error	Average Percent Error
1	-12.4	-12.7	-2.3%	-3.3%
2	-11.9	-12.4	-4.3%	
3	-11.3	-12.0	-5.6%	
4	-10.8	-10.9	-0.7%	

The third member was the first member fabricated where the core was fractured along the base, removing the possibility of any other stresses affecting the full release of the strain. The mix for this member had a modulus of elasticity of 2.17 GPa and an average compressive strength of 30.9 MPa. Table 5.3 shows the applied stress, the stress immediately after coring, and the stress once the core was fractured. The percent error was calculated between the applied stress and the re-bounding stress with the core removed. The percent errors were more consistent and slightly more accurate than the previous two test members.

Table 5.3 Beam 3 Stress Values and Corresponding Errors

Strain Gage	Applied Stress (psi)	"Re-bounding" of Stress with Core Attached to Member	"Re-bounding" of Stress with Core Removed	% Error with Core Removed	Average Percent Error
1	-13.6	-15.4	-13.6	-0.1%	-0.3%
2	-13.6	-16.8	-14.1	-3.1%	
3	-13.6	-15.7	-13.3	2.3%	

To verify the process of removing the core, one more member was created and the results are shown in table 5.4. Similar results were found compared to the third beam; thus, the results from the fourth beam confirmed the method used in the third beam. All the readings were within a reasonable tolerance to the actual values. The first gage had a large error, which could be due to the application of the gage to the member, causing the gage to drift; this margin of error demonstrates why multiple gages should be mounted to confirm the actual stress.

Table 5.4 Beam 4 Stress Values and Corresponding Errors

Strain Gage	Applied Stress (MPa)	"Re-bounding" of Stress with Core Attached to Member	"Re-bounding" of Stress with Core Extracted	% Error with Core Removed	Average Percent Error
1	-13.8	-14.7	-12.5	9.6%	1.7%
2	-13.8	-17.7	-13.9	-0.4%	
3	-13.8	-17.3	-13.9	-0.7%	
4	-13.8	-16.9	-14.1	-1.7%	

5.5 Conclusions from Post-Tensioned Tests

The post-tensioned members showed excellent correlation between the applied stresses and the stress determined from the surface strain relief method. When the core was fractured along its base, a higher level of accuracy was determined. At each location of the four members tested, the effect of a shearing stress was present at the bottom of the core, creating a positive stress at the surface and affecting the accuracy of the method. When the core was fractured at the base, any influence from the existing member was removed, allowing for a full stress release of the core.

Chapter 6 Implementation and Recommendations

The research showed highly accurate results using a 25.4 mm strain gage and coring around the gage. To continue the project in hopes of determining in situ stresses in a prestressed bridge girder, further testing is needed on members with varying cross sections and reinforcements at varying depths. From Phase I, when the core was removed a higher level of accuracy was found and this process should be continued. More research needs to be conducted to determine the effects of both specimen shape and aggregate size on the accuracy of the member. This will be evaluated in Phase 2 of this study.

In order to determine the prestressing force in the members with pre-tensioned strands, a crack-opening procedure such as the one used by Pessiki et al. (1996), Larson et al. (2005), and Peterman (2007) will be employed in Phase 2 and compared with the results obtained using the surface-strain relief method established in Phase 1.

Chapter 7 Final Conclusions

The surface strain relief method provided a high level of accuracy when coring to a depth of 25.4 mm with a 25.4 mm linear strain gage and a 63.5 mm diamond core bit. The data confirmed these conclusions. The results showed that varying stress levels still produce high levels of accuracy. To obtain a full stress release, the cores were fractured along the base of the core, and the core removed, releasing the core from the member and removing the influence of the member on the stress of the core. The development of the process worked well when testing all of the members. Rather than water, a vacuum was used to remove the dust when coring. Eliminating water reduced of the possibility of damage to the gage or the introduction of additional strains produced by the concrete absorbing the water.

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Appendix A:

Table A.1. Beam 1: Strain values before and after testing

Strain Gage	Initial Reading	After 1 Hour	After 2 Hours	After 3 Hours	After 4 Hours	Reading Before Coring	Reading After Coring	Change in Strain Before Breaking Out Core	Depth of Core (mm)
1	-481	-526	-539	-544	-544	-544	91	-635	24.4
2	-471	-	-	-	-	0	605	-605	24.4
3	-442	-482	-496	-502	-512	-514	94	-608	25.4
4	-486	-517	-522	-521	-523	-519	108	-627	26.4

Note: Average Modulus of Elasticity = 2.38 GPa

Table A.2. Beam 2: Strain values before and after testing

Strain Gage	Initial Reading	After 1 Hour	After 2 Hours	After 3 Hours	After 4 Hours	Reading Before Coring	Reading After Coring	Change in Strain Before Breaking Out Core	Depth of Core (mm)
1*				0	-62	-62	573	-635	26.4
2	-509	-562	-586	-601	-615	-609	50	-659	25.4
3	-522	-578	-598	-621	-634	-631	19	-650	25.4
4	-461	-510	-532	-542	-548	-548	31	-579	24.9

Note: Average Modulus of Elasticity = 2.21 GPa

* The first strain gage malfunctioned and was repaired after the load was applied so the gage was set to zero and then cored.

Table A.3. Beam 3: Strain values before and after testing

Strain Gage	Initial Reading	After 1 Hour	After 2 Hours	After 3 Hours	After 4 Hours	Reading Before Coring	Reading After Coring	Reading After Breakout	Change in Strain Before Breaking Out Core	Change in Strain After Breaking Out Core	Depth of Core (mm)
1	-435	-473	-466	-451	-438	-438	275	192	-713	-630	25.4
2	-525	-592	-609	-616	-624	-620	154	29	-774	-649	25.4
3**	-565	-633	-646	-648	-651	-	-	-	-	-	-
4	-500	-551	-554	-546	-538	-515	208	100	-723	-615	25.1

Note: Average Modulus of Elasticity = 2.17 GPa
** The third strain gage was damaged during coring.

Table A.4. Beam 4: Strain values before and after testing

Strain Gage	Initial Reading	After 1 Hour	After 2 Hours	After 3 Hours	After 4 Hours	Reading Before Coring	Reading After Coring	Reading After Breakout	Change in Strain Before Breaking Out Core	Change in Strain After Breaking Out Core	Depth of Core (mm)
1	-329	-369	-380	-400	-581	-588	40	-46	-628	-542	25.1
2	-498	-571	-608	-625	-302	-284	473	318	-757	-602	24.4
3	-462	-519	-545	-573	-607	-621	122	-17	-743	-604	24.4
4	-504	-564	-579	-601	-622	-626	100	-16	-726	-610	24.4

Note: Average Modulus of Elasticity = 2.30 GPa