Design and Behavior of Precast, Prestressed Girders Made Continuous –
An Analytical and Experimental Study

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Over the past fifty years, many states have recognized the benefits of making precast, prestressed multi-girder bridges continuous by connecting the girders with a continuity diaphragm. Although there is widespread agreement on the benefits of continuous construction, there has not been as much agreement on either the methods used for design of these systems or the details used for the continuity connections.

To aid designers in choosing the most appropriate method, an analytical and experimental study was undertaken at Virginia Tech. Analyses were done to compare the differences in the predicted continuity moments for different design methods and assumptions over a range of commonly used systems of Precast Concrete Bulb Tee (PCBT) girders and cast-in-place slabs. The results of the analyses were used to develop three continuity connection details for testing during the experimental study. Three different continuity connections were tested using full depth PCBT 45 in. deep girders made continuous with a 6 ft wide slab.

The bottom of the ends of the girders were made continuous with the continuity connection by extending prestressing strands for one test and extending 180 degree bent bars for the other test. Both connections adequately resisted service, cyclic, and ultimate loads. But, the test with the extended bars remained stiffer during cyclic loading and is recommended for use. A third test was performed on a system using only a slab cast across the top of the girders. Two primary cracks formed above the ends of the girders at the joint during service testing, after which no significant increase in damage took place.

Results from the analytical study indicate that the predicted positive thermal restraint moments may be significant, similar in magnitude to the actual positive cracking moment capacities. Results from the experimental study indicate that restraint moments develop early due to thermal expansion of the deck during curing and subsequent differential shrinkage; however, the magnitudes of the early age restraint moments are much less than conventional analyses predict.
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1 Introduction

Beginning in the late 1950’s in the United States, the advantages of making a multi-span, simply supported prestressed I-girder bridge continuous by connecting the ends of the girders over the supports with a continuity diaphragm connection began to be investigated. Early in this process, the Portland Cement Association recognized that continuity could be beneficial in three distinct ways (Kaar, et al 1960). First, continuity over a support could reduce both the deflections and maximum moments at the mid-span of a girder. This could allow for fewer strands to be used (approximately 5 to 15 percent fewer strands) or for longer span lengths to be achieved (Freyermuth 1969). Second, the elimination of the joints over the supports could improve the long term durability of a girder by reducing the amount of water and salt that potentially would cause deterioration of the concrete. Also, the elimination of the joints could provide for an improved riding surface. Third, the continuity could help to provide reserve load capacity in the event of an overload condition.

Over the past fifty years, many states have recognized these benefits and are designing their precast, prestressed girder bridges as continuous. Although there is good agreement on the benefits of continuous construction, there is not as much agreement on either the methods used for design of these systems or the details used for the continuity connections. The purpose of this study is to obtain a better understanding of both the design and behavior of precast, prestressed girders with cast-in-place slabs made continuous. To achieve this purpose, an initial analytical study was undertaken to compare the prediction of restraint moments with the most commonly used current design methods. Restraint moments due to time dependent effects such as creep and shrinkage as well as restraint moments due to thermal gradients were considered. Results from the analytical study were then used to propose three continuity connections for testing. Full depth girders sections were fabricated and shipped to Virginia Tech where the girders were connected with a continuity connection and tested for service, cyclic, and ultimate load capacities.

1.1 Overview of the Problem

In comparison to a composite girder-slab system made up of simple spans, it is more difficult to predict the behavior of a similar system when it is made continuous with a continuity connection because the structure undergoes time dependent changes. When a simple span
precast, prestressed girder is initially placed on its supports, the prestressing force causes the concrete to creep. For typical simple span structures, this causes an upward deflection to occur. Once the cast-in-place slab is placed, an initial downward deflection occurs. Over time, creep effects will generally cause additional deflections to occur. Since the girders and the deck are usually made of concretes having different strengths and are cast at different times, differential shrinkage may occur within the composite system. This differential shrinkage may also cause deflections to occur. The net deflection of the system due to these factors (and others) must then be predicted. After the continuity connection is made, any deflections in the system will cause rotations over the supports which cause the development of restraint moments and shears in the ends of the girders at the connection because the system is now indeterminate. Accounting for all of the possible factors is difficult, and has resulted in several different design methods to predict the behavior of these systems.

In Virginia, precast, prestressed girders made composite with a cast-in-place deck have been commonly used for highway construction since the 1960’s. In the earlier days of their use, many bridges were designed as simple spans. The issue of predicting restraint moments did not exist, since a joint was provided instead of a continuity connection. Throughout the years, it was found that simply eliminating the joints in a bridge could help to reduce the maintenance costs throughout the life of the structure. Today, many bridges in Virginia are made continuous, requiring the prediction of the restraint moments.

Recently, the Virginia Department of Transportation (VDOT) began to use a different standard shape girder for typical highway bridges, the Precast Concrete Bulb-Tee (PCBT) shape. To gain the full advantages of the new shape, VDOT desired to investigate the continuity detail for this newer shape. In October 2001, Professors Carin Roberts-Wollmann and Tommy Cousins proposed a Research Project entitled “Development of an Optimized Continuity Diaphragm for New PCBT Girders” to VDOT to study this connection. This proposal (see Appendix A), was accepted, providing the basis for this research.

VDOT provided standard “Prestressed Concrete Beams Standard Bulb-T Details and Section Properties” and “Bulb-T Preliminary Design Tables”, both of which have been used for the basis of this research. (See Appendix B) A literature review concentrating on the current state of practice was performed and a series of analytic studies was undertaken to investigate the
typical girder and slab arrangements provided by VDOT. Based on this preliminary work, it was found that there are many factors which influence both the behavior and the design process of girders made continuous. In fact, simply choosing one design method versus another can provide significantly different predictions for the restraint moments.

1.2 Overview of Factors Influencing Behavior

The main reason that there is not better uniformity in the design of girders made continuous is the fact that there are so many variables which influence the behavior of the system. Unlike some structural systems which have properties that remain relatively static over time, a system of girders made continuous is influenced by many factors that cause the properties of the system to change over time. The primary material for the girders and the diaphragm, concrete, has properties that change dramatically in the early stages of hydration and continue to change years after the final set has occurred. In fact, the word concrete comes from the Latin word *concretus* which means to grow (Mehta 1986). The addition of the prestressing steel to the system introduces additional material and time dependent influences on the behavior of the system, making the behavior of the system more difficult to predict.

Most precast, prestressed girders used for bridge construction are non-composite for self-weight and the dead load of the deck but are later made composite by the addition of a cast-in-place concrete deck. Therefore, both composite and non-composite properties must be considered in the design process. Also, unique to this type of construction, the construction sequencing can have a large influence on the behavior of the system.

Once a multi-span system is made continuous, thermal restraint moments will develop and induce additional stresses in the system. Some design methods do not consider these influences; however, the stresses caused by the thermal restraint moments may be significant and have been considered in this study.

Considering just some of these influences, it can be seen that the behavior of a system made of precast, prestressed concrete made continuous can be difficult to predict. As some responses begin to influence the behavior of other material properties, the validity of using the law of superposition begins to be questioned, making the prediction of the behavior of these systems even more difficult.
1.3 Overview of the Current Design Process

In the United States, there have been two main design procedures used in the past for predicting the required design moments at the continuity connections, the PCA Method (Freyermuth, 1969) and the National Cooperative Highway Research Program (NCHRP) 322 Method (Oesterle, et al 1989). Currently, a third document has been developed under the NCHRP 12-53 research. This research has been published as NCHRP Report 519 (Miller, et al 2004). A recent survey performed for the NCHRP 519 Project revealed that about 35 percent of those responding use the PCA Method while about 9 percent use the NCHRP 322 method (Hastak et al, 2003). In addition, the survey revealed that about 48 percent of those responding use some type of standard detail for the positive moment connection while about 25 percent do not use any positive moment connection. Clearly, differences in the design process exist throughout the country. Regardless of the differences, the single goal of the design process is to predict the moments at the continuity support so that the appropriate amount of steel can be provided for both the positive and negative moments for both service and ultimate loads.

When both of the current methods are used to predict the restraint moments of the same system, the predicted moments often differ significantly for girder and slab systems commonly used in highway construction. It is commonly believed that the PCA Method usually predicts a higher positive moment than does the NCHRP 322 method.

A designer is then faced with a difficult decision: which method should be used for the design? One procedure that practicing engineers use is to design using both methods and use the method that is most conservative. However, always using the most conservative method can at times produce designs with so much positive moment steel that the section can not be easily constructed. On the other had, recommending a less conservative design is sometimes difficult if not impossible for a practicing engineer to do without assurances that the method producing the design is backed by sound engineering. Therefore, it is desirable to understand what influences the different design methods and why they can at times predict significantly different moments.

1.4 Scope of Study

An analytical study was performed and summarized in a paper entitled “Influences of Design Methods and Assumptions on Continuity Moments in Multi-Girder Bridges,” a copy of which is included in Appendix C. From the analytical study, three continuity connections were
developed for testing. The first two continuity sections tested had a full continuity diaphragm with a cast-in-place deck. To connect the bottom of the girders to the diaphragm in tension, Test 1 used extended prestressing strands extending from the bottom of the girders and bent at a 90 degree angle while Test 2 used extended 180 degree No. 6 U bars extending from the bottom of the girders. The third continuity connection tested, Test 3 consisted of a slab only, cast continuous over the girders. At the fabricator, six full depth PCBT-45 girders, each 15 ft - 4 in. long, were instrumented and monitored during steam curing and detensioning. Concrete cylinders and beam specimens were made to determine material properties of the concrete. The girders were then shipped to Virginia Tech where the girders for each test were connected with the different continuity diaphragms. The continuous systems were monitored throughout all stages to obtain a better understanding of the early age development of restraint moments. After the concrete decks had reached 28 day strength, service moments were applied to the continuity diaphragm connections to determine the initial cracking moment capacity. Cyclic loads were then applied to determine how much the capacity of the connections degraded over time. Finally, the sections were loaded to near ultimate conditions to determine the ultimate moment capacities.

1.5 Organization of this Document

The body of this document begins with Chapter 1, Introduction. Following this introductory chapter, Chapter 2 presents a Literature Review of the past and present body of knowledge pertaining to continuous girders. Chapter 3 presents the Research Methods and Materials used in the experimental testing. Chapter 4 presents the Laboratory Results and Discussions for the material testing. Chapter 5 presents the work performed during the curing process in Fabrication Phase Results and Discussions. Chapter 6 presents the results of the studies performed when the decks were cast in the Static Phase Results and Discussions. Chapter 7 presents the testing results done after the decks reached design strength in the Service and Cyclic Phase Results and Discussions. The Conclusions and Recommendations are presented in Chapter 8.
2 Literature Review

2.1 Early Studies

2.1.1 PCA

One of the first major studies performed to address the issues of prestress girders with a cast-in-place deck made continuous was performed in the Research and Development Laboratories of the Portland Cement Association and was published in a series of Development Department Bulletins beginning in 1960. Bulletin D34 summarized the initial pilot tests which were designed to investigate the strength of the continuity connection in negative bending as well as the strength and moment redistribution of the continuous girders with a negative moment connection (Kaar, et al 1960). In this pilot study, fifteen half-scale specimens divided into three groups were tested in negative bending. The specimens were designed to resemble a 44 in. deep AASHTO type girder. The half scale specimens were 22 in. deep and had a 2 ft wide by 3 in. deep composite deck. The percent of steel in the girders included 0.0, 0.6 and 0.9 percent. In the deck, the percentages included 0.83, 1.66 and 2.49 percent. The concrete strengths ranged from 3210 psi to 4390 psi for the deck and from 4840 psi to 5950 psi for the girders.

Testing took place at girder ages of about fifteen days and deck ages of about seven days. The failure of the sections always took place outside of the region of the diaphragm, and it was concluded that a negative moment connection was “fundamentally sound”. It was found that the ultimate negative moment was governed by tension in the deck reinforcing for deck reinforcing percentages ranging from 0.5 to 1.5 percent. In this range, it was recommended that the effects of the prestressing on the girder could probably be neglected for design provided the effective prestress in the concrete was less than 2000 psi. For the 5000 psi test specimens, it was later recommended that the effects of prestressing could be neglected in the design of the negative moment provided the maximum precompression stress in the girders was less than 0.4*f'_c and that the continuity reinforcing in the deck was less than 1.5 percent. It was also concluded that moments would redistribute sufficiently within a practical range of deck reinforcing percentages (Kaar, et al 1960).

Continuing the work which was published in Development Department Bulletin D34, additional tests were performed and published later that year in Bulletin D43 (Mattock, et al 1960). An additional test of the negative moment capacity of a half-scale two span continuous
A girder was performed. Scale effects were compensated for by hanging dead load blocks along the girder to simulate the actual anticipated stresses. The system was tested in negative bending over the support by first loading to its design negative moment composed of the Live Load plus Impact as determined by the ACI-ASCE Joint Committee 323, “Tentative Recommendations for Prestressed Concrete (1958),” and the Seventh Edition of the “AASHTO Standard Specifications for Highway Bridges”. Using ultimate design, the design load case was taken as \((1.5D + 2.5LL*I)\). Therefore, for the negative design moment, the load case for the two span unit was simply \(2.5LL*I\). It was then unloaded and reloaded to two times the design load. After this loading was removed, the specimen was loaded to failure.

It was concluded that a negative moment connection composed of deformed reinforcing in the deck would perform under both service and ultimate load conditions. The connection was also considered satisfactory for continuity, approaching the elastic theory continuity relationship for low loadings and approaching the limit state theory of continuity for the ultimate loads. Deflections and crack formations were also considered acceptable.

A second series of tests using seven half-scale stub specimens was then performed to investigate the fatigue capacity of the negative moment connection. The ultimate moment capacity of the section was determined to be 202.3 kip-ft. A design moment, \(M_d\), of 80.92 kip-ft which was 0.4 times the ultimate moment (1/ the load factor of 2.5) was considered for testing. Dynamic testing took place by applying a load ranging from a minimum value of \(0.28*M_d = 56.64\) kip-ft to maximum values that ranged from \(1.5M_d\) to \(2.61M_d\). The minimum value was required to keep the ram in contact with the specimen. The pulsator ram applied the loads at a rate of 250 times per minute. Results from these fatigue tests are shown in Table 2.1.

It was concluded that the fatigue strength of the connection was limited to the fatigue strength of the reinforcing steel in the deck. Provided that actual moments are kept within the design limits (less than \(M_d\), then it was concluded that the connection could be expected to withstand an infinite number of load applications. Cracking and changes in the flexibility of the connection were also found to be acceptable.

A third series of tests was performed to address the possibility of positive moments developing at the continuity connection. These positive moments were believed to potentially have magnitudes ranging from fifteen to twenty-five percent of the negative live load moments.
Table 2.1 – Results of Negative Moment Fatigue Testing

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Maximum Moment</th>
<th>Number of Load Applications</th>
</tr>
</thead>
<tbody>
<tr>
<td>J1</td>
<td>1.5M_d</td>
<td>Discontinued at 10,430,000</td>
</tr>
<tr>
<td>J2</td>
<td>1.72M_d</td>
<td>2,809,000</td>
</tr>
<tr>
<td>J3</td>
<td>1.95M_d</td>
<td>641,000</td>
</tr>
<tr>
<td>J4</td>
<td>2.17M_d</td>
<td>127,000</td>
</tr>
<tr>
<td>J5</td>
<td>2.39M_d</td>
<td>52,000</td>
</tr>
<tr>
<td>J6</td>
<td>2.61M_d</td>
<td>30,000</td>
</tr>
<tr>
<td>J7</td>
<td>2.41M_d to Yield</td>
<td>Static Test to Failure</td>
</tr>
<tr>
<td></td>
<td>3.54M_d to Ultimate</td>
<td></td>
</tr>
</tbody>
</table>

Although it was believed that positive moments could only be produced for continuous spans of three or more, a two span unit was tested for simplicity. Seven half-scale specimens were tested, three of which used an angle welded to mild reinforcing projecting from the bottoms of the girders to achieve the connection. The other four used extended bars bent at a 90 degree angle and lapped within the diaphragm to achieve the connection. Five of the connections were tested in static loading conditions and the other two were tested in dynamic loading conditions. The static loading was performed by removing the support under the diaphragm and applying a vertical load downwards at the top of the diaphragm to create the positive moment. Failure of the diaphragms with the welded bar connections in static loading occurred by crushing of the concrete at the top of the diaphragm. Failure of two of the bent bar connections occurred by a brittle fracture of the reinforcing at the knee of the hook. The other connection (R5, which obtained the highest strength among the bent bar type connections) failed by crushing of the concrete within the hooks along with spreading out of the hooks. The fatigue loading was designed to apply a range of 20 ksi in the reinforcing steel. A minimum loading was applied due to the removal of the supports and contact requirements of the ram. The actual maximum load tested was 0.55 times the ultimate capacity and the stress range in the steel reinforcing varied from 7.8 ksi to 28.1 ksi. During the fatigue testing, the bent bars failed by fracture at the knee of the bend. A summary of the specimens tested and the results are shown in Table 2.2.
It was concluded that the welded type connection would perform satisfactorily for both service and ultimate loading conditions. It was also concluded that the hook bar connections, which did not perform as well, would possibly perform satisfactorily provided that some adjustments were made to the detailing of the hooks. First, it was recommended that the radius of the bend for the hooks be increased to at least the diameter of the bar. The actual bend radii used were not provided in the literature; however, it can be assumed that it was less than the diameter of the bar. Secondly, it was recommended that the distance from the end of the girder to the inside face of the hook bar be increased to a minimum of at least twelve times the bar diameter.

Another part of the studies undertaken by the Portland Cement Association was an investigation into the effects of creep and shrinkage on continuous bridges. This study, which monitored two half-scale two span continuous girder systems over a period of approximately two years, was presented in Development Department Bulletin D46 (Mattock 1961). The two girder systems investigated were similar except that girder 1/2 had no positive moment connection while girder 3/4 had a positive moment connection comprised of 4 No. 3 hook bars projecting from the ends of each girder. To account for the effects of scaling, large dead load blocks were hung from the structure throughout the testing. Measurements were made of the center support reactions, midspan deflections, concrete strains, and strains in the reinforcing steel over the interior support. To measure the strains in the reinforcing over the supports, a pocket was made in the concrete exposing the reinforcing steel. Holes were then drilled in the reinforcing steel at two locations and the changes in distances between these points were measured with a

![Table 2.2 – Results of Positive Moment Testing](image)

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Reinforcing Type</th>
<th>M_{test}/M_{calc}</th>
<th>No. Cycles to Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1</td>
<td>2 No. 6 Welded</td>
<td>1.62</td>
<td></td>
</tr>
<tr>
<td>R1A</td>
<td>2 No. 6 Welded</td>
<td>-</td>
<td>1,018,000</td>
</tr>
<tr>
<td>R2</td>
<td>3 No. 4 Welded</td>
<td>1.47</td>
<td>-</td>
</tr>
<tr>
<td>R3</td>
<td>2 No. 6 Hook Bars</td>
<td>0.69</td>
<td>-</td>
</tr>
<tr>
<td>R3A</td>
<td>2 No. 6 Hook Bars</td>
<td>-</td>
<td>670,000</td>
</tr>
<tr>
<td>R4</td>
<td>3 No. 5 Hook Bars</td>
<td>0.88</td>
<td>-</td>
</tr>
<tr>
<td>R5</td>
<td>3 No. 4 Hook Bars</td>
<td>0.99</td>
<td>-</td>
</tr>
</tbody>
</table>
Whittemore gage. Ten foot control sections of the girder and the slab were cast separately and monitored for free shrinkage.

After casting the deck and diaphragm connection, the two specimens were observed and tested occasionally to determine the degree of continuity being provided by the connection. Initially, the center reaction increased (became more compressive) for approximately forty days. An increase in the reaction would indicate the creation of a negative moment in the connection. After this time, the reaction began to decrease until it began to indicate a net decrease in the center reaction for both specimens, indicating the development of a positive moment in the connection. At an age of about 337 days, girder 1/2, which had no positive moment steel, displayed cracking in the bottom of the connection. At this point, the positive restraint moment remained almost constant while the positive restraint moment continued to increase for girder 3/4. After nearly two years, the girders were tested to failure and compared with a similar continuous girder system, girder 7/8, which was tested to failure only 12 days after casting the deck. All three girders were found to have similar ultimate capacities. However, girder 1/2 and girder 3/4 reached slightly higher ultimate loads than girder 7/8. It was concluded that the restraint moments and deformations due to creep and shrinkage did not influence the ultimate strength of the system. The slight improvement shown by girders 1/2 and 3/4 was attributed to the higher concrete strengths due to the age of the concrete.

The distribution of stresses on the composite section was compared to two different methods used to predict the effects of creep and differential shrinkage, the Effective Modulus Method and the Rate of Creep Method. The Effective Modulus Method replaces the elastic modulus, \( E \), with a reduced secant modulus that accounts for the elastic and creep strains. Therefore, the effective modulus, \( E' \), can be represented by the following:

\[
E' = \frac{E}{1 + \phi}
\]  

(2.1)

where \( \phi \), the creep coefficient, is the ratio of the creep strain, \( \varepsilon_c \) to elastic strain, \( \varepsilon_e \).

The Rate of Creep Method assumes that the total creep can be determined by integrating the product of the rate of creep and the stress in the member over a given time. The creep can then be calculated by the following:
\[
\text{Creep} = \int_0^T \frac{f}{E} d\phi \ dT
\]  \hspace{1cm} (2.2)

where \( f \) is the stress, \( T \) is the time, and \( d\phi \) equals:

\[
d\phi = d\frac{\varepsilon_c}{\varepsilon_e} = Ed\varepsilon_c
\]  \hspace{1cm} (2.3)

For the test specimen that had the positive moment connection, girder 3/4, both methods
were used to compare the calculated change in the support reaction to the actual measured
change. When the two methods were plotted against the actual measured values, both showed a
similar trend. Both overestimated the effects at the early stages following the placing of the
continuity diaphragm, predicting an increase in the support reaction approximately 1.5 to 1.9
higher than actually observed. This indicates that the effects of differential shrinkage were
overestimated in the early stage. Since differential shrinkage causes a negative moment over the
support, the actual negative moment measured was less than both methods predicted. It was
concluded that the creep in the deck was probably more than calculated, since the creep
properties for the girder were used for both the deck and the girder. Allowing more creep to
occur in the deck in the calculations would allow for a decrease in the amount of differential
shrinkage, thus reducing the calculated support reaction and in turn reducing the negative
moment. Even though both of the methods overestimated the negative moment at the early
stage, it was concluded that there is no real need to know these values at the early stages, since
the stresses will only be temporary.

More importantly, both methods appeared to predict the observed decrease in the support
reaction at the later stage of observation quite well. The decrease in the support reaction
indicated that a positive moment was being developed in the continuity diaphragm. It was
concluded that both the Rate of Creep Method and the Effective Modulus Method could be used
for prediction of the positive moments induced by the combined effects of creep and shrinkage
(Mattock 1961).

2.1.2 PCA Method

In 1969, the Portland Cement Association released an engineering bulletin which was
based primarily on the earlier research documented in the Development Department Bulletins
released in the early 1960’s. This engineering bulletin, the “Design of Continuous Highway Bridges with Precast, Prestressed Concrete Girders”, became the standard for continuous design, and is still used by a considerable number of designers in the year 2004, thirty-five years later (Freyermuth 1969). The design guide contains two parts, the first part gives guidance on the procedures necessary in continuous design and the second part contains a design example.

The method gives guidance on how to determine the magnitude of any restraint moments that may develop over an interior support due to creep and differential shrinkage. First, the ratio of the creep strain to the elastic strain, $\Phi$, is determined for the girder. To obtain this value the specific creep value for a loading at 28 days is obtained from a graph using the elastic modulus of the girder concrete at the time of loading and assuming that the ultimate creep occurs at 20 years. This specific creep value is then adjusted for the age when the loading actually takes place. For a prestressed girder, this is the age of release of the strands, usually 1 or 2 days. The value is also adjusted for the actual volume to surface area ratio of the girder. Then, the amount of creep that has taken place is determined by entering a graph with the age that the continuity connection is made and determining the proportion of creep that has taken place. The remaining creep to take place is then determined by subtracting this value from one. The adjusted specific creep is multiplied by this value to give the ultimate creep expected. Finally, multiplying this value by the elastic modulus gives the ultimate creep coefficient, $\Phi$.

The basic shrinkage restraint moment due to the differential shrinkage that will take place between the deck and the girder is then determined. The basic shrinkage moment, $M_s$, is calculated from:

$$M_s = \varepsilon_s E_b A_b \left( e'_2 + \frac{t}{2} \right)$$  \hspace{1cm} (2.4)

where $\varepsilon_s$ is the differential shrinkage strain, $E_b$ is the modulus of elasticity for the concrete in the deck, $A_b$ is the area of the deck, $e'_2$ is the distance from the centroid of the composite section to the bottom of the deck, and $t$ is the deck thickness.

Two methods of determining the differential shrinkage strain are given. First, if test results are available for the mix to be used then a shrinkage time curve can be used for determining the remaining differential shrinkage strain. Since most designers do not have this information available, a second method is given which assumes an ultimate value for the
shrinkage and adjusts this value for the expected conditions. An ultimate shrinkage value of 0.600x10^{-3} is recommended for an exposure of fifty percent relative humidity. This value is then adjusted for the actual relative humidity expected by using a humidity correction factor. The adjusted ultimate shrinkage value is then multiplied by a factor accounting for the proportion of shrinkage that has taken place in the girder from the time the girder was cast to the time the deck was cast. This factor comes from the same graph used to determine the proportion of creep which has taken place. It is assumed in the procedure that the girder and the deck will have the same ultimate shrinkage values. Therefore, the amount of shrinkage that will cause differential shrinkage will be the amount of shrinkage that has already taken place in the girder when the deck is cast.

Once the shrinkage restraint moment is determined for a given span, the basic or unadjusted restraint moments due to shrinkage, dead load creep, and creep caused by the prestressing force can be determined. The moment distribution method is used to determine the resulting moments for multiple span situations. The equivalent simple span moments are applied to each span and the resulting restraint moments are determined using the moment distribution method. The resulting restraint moments are not added back to the equivalent simple span moments. Instead, the resulting restraint moments are then adjusted due to time effects and are used for the design of the diaphragm.

The rate of creep method is used to account for the influence that time has on the creep of concrete. Loads that are applied at an initial time and do not change such as the dead loads and the prestress force are multiplied by the quantity (1-e^{-\phi}). Loads that are initially zero and increase slowly over time such as the differential shrinkage are multiplied by the quantity (1-e^{-\phi})/\phi. The restraint moments due to dead loads, prestress force, and differential shrinkage are then summed up to determine the total restraint moment. For a typical structure, the dead load and shrinkage restraint moments will cause a negative restraint moment to develop while the restraint moment due to the creep caused by the prestressing force will cause a positive restraint moment to develop over the interior supports.

The design procedure outlined in the PCA method makes several assumptions. The influence of the length of the diaphragm in the longitudinal direction is not considered in the design. Essentially, a structure is modeled from centerline of bearing to centerline of bearing
with the support considered to be a single support. It is assumed that the deck and the girder will have the same ultimate shrinkage values. Also, the influence of prestress losses is not accounted for directly. Instead, the final force after all losses is used in the calculation of the restraint moment due to prestress force.

For the design of the negative moment reinforcing in the deck, it is recommended that the compressive strength of the girder concrete be used instead of the compressive strength of the diaphragm concrete. It is recommended that ultimate strength design be used for this region with load factors of 2.5 and 1.5 on Live Load and Dead Load, respectively. The compressive stress in the girder at the ends due to the prestressing steel may be neglected provided that the stress is less than 0.4 f’c and the continuity reinforcement index is less than 1.5 percent. A Rectangular Beam Curve is provided so that knowing the value of \( \frac{M_u}{bd^2} \), \( f_y \) of the steel, and the \( f'c \) of the concrete, the steel ratio, \( \rho \), for the required deck reinforcing can be determined.

In the design example provided for the method, it was found that high compressive stresses at the bottom of the girder near the end are calculated for the combination of negative live load and prestressing force. The value obtained is greater than the recommended 0.4 f’c. The results of the PCA tests indicated that these high stresses do not affect the ultimate strength of the section. The earlier PCA studies suggested that the presence of inelastic strains will offer stress relief. Also, the transfer length of the strands does not allow full compression to occur at the ends and the presence of the diaphragm acts to restrain the area thus increasing its allowable compressive strength. The design example also assumes that the deck and diaphragm are placed 28 days after the release of strands. It is likely that the use of a 28 day time period in this design example set a precedence that has caused many designers to continue to use 28 days as the required time period since then.

2.2 National Cooperative Highway Research Program

The National Cooperative Research Program (NCHRP) has sponsored two separate projects addressing continuity connections in multi-girder bridges. The first study, NCHRP 322, took place in the mid 1980’s. The second study, NCHRP 519, took place recently at the University of Cincinnati. Virginia Tech was given an advance Draft Final Report for use in the early stages of this study. A brief summary of the two reports is included.
2.2.1 NCHRP 322

In the mid 1980’s, the NCHRP undertook a project designed to address the behavior and design methods of precast, prestressed girders made continuous. This study was performed primarily at the Construction Technology Laboratories in Skokie, Illinois, and was released in 1989 as NCHRP Report 322, “Design of Precast Prestressed Girders Made Continuous” (Oesterle, et al 1989). This report investigated many of the assumptions and design requirements which were presented in the earlier 1969 PCA report. In an attempt to determine the current state of practice throughout the country, the project’s First Task included a literature review and a questionnaire. Also included in the First Task was a limited number of tests performed on steam cured concrete specimens to investigate creep and shrinkage specifically for steam cured precast, prestressed concrete with loadings at early age (two day release of strands). The Second Task included parametric studies performed using existing computer programs and two newly created programs to determine the influence that certain material properties and design assumptions would have on the resulting service moments. The Third Task included developing analytic procedures to investigate the flexural strength of the members along their lengths. The final task, the Fourth Task, concentrated on determining the strength and service requirements for the continuity connections over interior supports.

Results of the questionnaire indicated that of the 42 respondents who provide positive moment reinforcement in the continuity connection, 30, or approximately 71 percent, used the PCA method as the primary method for design. Concerning the type of reinforcing used for the positive moment connection, 18 respondents used embedded bent bars and 21 respondents used extended strands, a 46 to 54 percent split. A listing of bridge performance problems was compiled from the questionnaire. Some of the reported problems included: positive moment reinforcing requiring field adjustment, extended strands accidentally cut off, transverse cracking of the deck above the negative moment region, incorrect construction sequence, cracking of the continuity diaphragm due to long term creep and shrinkage, and cracking and spalling of the continuity diaphragm when cast prior to the deck.

The literature review found that there was little information concerning the prediction of creep and shrinkage of steam cured concrete. Five creep and shrinkage tests were performed to try to increase the understanding of the prediction of creep and shrinkage specifically of steam cured concrete. The different concretes used for the tests were provided by one local and three
non-local precasters. Standard 6 in. by 12 in. cylinders were cast and allowed to steam cure at the plants and then were shipped to the CTL laboratories as quickly as possible for testing. The creep testing was done in accordance to ASTM Standard C 512 and the drying shrinkage was measured from a control cylinder stored in the same environment as the cylinders used for the creep tests. The creep strain was measured as the total strain minus the shrinkage strain. The creep coefficient was measured as the creep strain divided by the initial elastic strain. The results were compared to the predicted results of ACI-209, “Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures” (1982).

It was found that the actual creep and shrinkage values measured were within the ranges predicted by ACI-209 for the later ages of the concrete. However, for the early stage of the concrete, the ACI-209 predictions were not as accurate. For concrete less than 15 days old, the actual shrinkage strains for all five specimens were greater than the ACI-209 recommended upper bound. Also, the ACI-209 recommended ultimate creep coefficients were found to be less than the actual measured creep coefficients for four of the five tests. The results were also compared to the simplified Bazant-Panula (BP2) prediction model. This model, which is not intended to be used for concretes loaded earlier than seven days, resulted in larger errors in the predicted values than the ACI-209 model. Therefore, the ACI-209 model was used for the remainder of the study.

The program PBEAM (Oesterle, et al 1989) was chosen to perform the analysis of the continuous structures. The program uses a step by step analysis method to account for the nonlinear stress-strain response of the concrete. The ACI-209 model is used by PBEAM to estimate the time dependent factors such as strength, creep, and shrinkage. Construction sequencing was modeled by allowing the input of different ages of continuity. The program also modeled crack development and was able to track whether a crack in the concrete was open or closed. At the cracked sections, a “tension stiffened” effective stress-strain model for the reinforcing (both top and bottom) at the support was used. The program PBEAM was used to perform the parametric studies for the project; however, it was decided that the program was too cumbersome to use for typical design applications. To help facilitate the design process, a new program, BRIDGERM, was written. In essence, BRIDGERM was written to perform the same function of PBEAM, but in a limited way. BRIDGERM was written primarily to help determine the restraint moments, both positive and negative, that may develop in a continuous member.
The original version of BRIDGERM, which was written in FORTRAN, was later rewritten in Visual Basic and is now called RMCALC (McDonagh 2001). The code for RMCALC is essentially identical to the code for BRIDGERM.

A second program, WALL_HINGE, (Oesterle, et al 1989) which was originally developed to analyze concrete shear walls, was also used to model the behavior of the structure at the continuity connections at their failure loads. WALL_HINGE considers the influence of strength and inelastic deformation capacity over the hinge region under combined loads. A tri-axial concrete stress-strain distribution with the strength criteria based on the octahedral shear stress was used by the program to equate the summation of the tensile strains over a length at the tail of the fan region to the summation of the compressive strains over a much smaller length at the tip of the fan region.

In conjunction with the program WALL_HINGE, a program called BEAM BUSTER (Oesterle, et al 1989) was used to model the moment-curvature relationship of the system. The three girders previously tested in the PCA tests were analyzed by these two programs and were found to theoretically fail in a similar manner to that which was observed.

Extensive parametric studies were performed using the above programs to determine the influence of many different parameters on the time dependent restraint moments and the ultimate moments in the continuous system. The results of these parametric studies were used in conjunction with the results of the questionnaire and the limited creep and shrinkage testing to form several observations and recommendations.

Considering the positive moment which may develop at an interior support, it was found that “…providing positive moment reinforcement has no benefit for flexural behavior of this type of bridge…” and that “…the provision of positive moment reinforcement at the supports is not recommended” (Oesterle, et al 1989, pg. 35). Although it was indicated that the positive moment reinforcing over a support can help to reduce the size of the cracks that may develop, the presence of the positive moment reinforcing was found to have a negligible effect on the positive moment at the midspan of the structure. Although providing positive moment reinforcing at an interior support would provide some benefit to the structure by introducing continuity for superimposed loads, this benefit was found to be offset by two factors. First, positive moment caused by the positive restraint moment above the support would cause an increase in the
positive midspan moment. Second, as a positive moment would develop over a support, cracking would occur in the bottom of the continuity region. The superimposed dead and live loads added to the structure would then have to cause sufficient rotation in the section to overcome the cracking before the benefits of continuity could be realized. In other words, due to the cracking in the bottom of the continuity connection, full continuity could not be established. Therefore, it was recommended that positive moment reinforcing not be used. Responses from the questionnaire indicated that some states had already used continuity connections with no positive moment reinforcing. Responses by California, Florida, Minnesota and Wisconsin all indicated that they had experience using continuous decks with no positive moment reinforcing and that they had not experienced any problems with this type of detail.

It was recommended that the program BRIDGERM be used for determining restraint moments. If specific concrete properties are not known during design, then it was recommended that the provisions of ACI-209 be used to determine the properties. A specific recommended age of continuity was not given; however, it was reported that most girders were made continuous when they were about 10 days to 90 days old. An early age of continuity of 17 days was used in the parametric studies.

Since it was recommended that positive moment reinforcing not be used at the interior supports, a program BRIDGEILL was developed to determine the actual distribution of live load moments on such a system. In effect, this program calculates the distribution of live loads from a truck or lane loading assuming that a hinge will occur if the positive moment occurs over an interior support.

Recommendations were provided for determining the magnitude of the service moments which should be used for design. First, considering the midspan positive service moments, it was found that the moment which should be used for design was a function of the magnitude of the continuity moment at the supports. For an interior span of a continuous structure, the restraint moments due to time dependent effects cause a constant moment to occur over the span which may be either positive or negative. The continuity moment at the supports is defined as the summation of the restraint moments and the moments due to additional dead load and live load plus impact acting on the continuous structure. The net continuity moment may be either positive or negative in value.
For a given span, if the average of the two continuity moments at the ends is positive, then the effective continuity has been reduced from 100 percent to 0 percent. Simple span moments for the additional dead load and the live load plus impact should be used for determining the midspan positive service moments.

For a situation where the average of the two continuity moments is negative and less than 125 percent of the negative cracking moment at the supports, then some benefits of continuity for additional dead load and live load plus impact does exist. It was recommended that the design midspan positive service moment be taken as the summation of the calculated restraint moment, the calculated midspan moment obtained from additional dead load on the continuous system assuming full continuity and the midspan moment due to live load plus impact obtained from the program BRIDGELL. For a situation where the average of the continuity moments exceeds 125 percent of the cracking moment at the supports, then it was recommended that the moment in excess of 125 percent times the cracking moment be redistributed to the positive moment at midspan. This is done by taking the moment diagram and shifting it by the difference of the actual continuity moment and 125 percent times the cracking moment.

For comparison to allowable service stresses given in AASHTO, it was recommended that non-composite section properties of the girder be used for determining stresses caused by the girder self weight, the weight of the deck and the prestress force. Composite section properties were recommend for determining stresses due to additional dead load, live load plus impact and time dependent restraint effects.

For the negative service moments at the supports, it was recommended that the effects of negative restraint moments be included in the service check even though AASHTO specified that only the effects of the prestressing and the negative bending due to live load be considered. It was recommended to use BRIDGERM to calculate the negative moments over the support due to time dependent effects. The age of continuity used for determining the negative time dependent moment should be the latest age expected during construction. According to the responses from the questionnaire, girder ages varied at time of continuity; however, most girders were no more than approximately 90 days old when continuity was made. It was found that the maximum negative moment in the continuity connection occurred approximately 50 days after continuity was established. It was recommended that the negative service moment be check at an age of 50
Considering the fatigue limits on the continuity connection, it was concluded that negative moments induced at early ages should not be used for fatigue checks since these moments are generally considered to be transient. Instead, it was recommended that the lower limit for the stress range for fatigue checks be obtained by determining the sum of the restraint moment caused by an age of continuity of at least 700 days and any moment due to additional dead load. The upper limit for the stress range should be based on the additional moments due to live load plus impact.

The program BRIDGELL was recommended for determining the maximum negative moments over the supports due to Live loads plus impact. If the moment over the support was found to be positive, then it was recommended to use the moment due to additional dead load and live load plus impact for determining the maximum negative moment over the support.

The authors recommended that time dependent effects should not be considered for the ultimate strength design of the members. As demonstrated in the earlier PCA tests, girders tested to failure demonstrated the same ultimate strength regardless of the presence of restraint moments. The authors state that “Stresses and strains induced in the girders by volume change from creep and shrinkage are self-limited within the girder in that they are relieved by the deformations accompanying cracking in the concrete and yielding of reinforcement. As a result, presence or absence of time-dependent restraint moments has no effect on the strength of the structure (Oesterle, et al 1989, pg. 38).”

For determining the maximum negative and positive moments to use for the strength design, it was recommended to use the program BRIDGELL for the Live Load plus impact moments. Additional dead loads to the composite section should be calculated using conventional methods assuming full continuity. Time dependent restraint moments should not be used in the strength design. For negative moments at the continuity connections, it was recommend that the reinforcement ratio of the continuous steel in the deck, $\rho$, be less than 50 percent of the balanced reinforcing ratio to provide for ductile behavior.

2.2.2 NCHRP 519

The project NCHRP 519, which has recently been completed, was a joint venture
between the University of Cincinnati and Ralph Whitehead Associates (Miller, et al 2004). The preliminary results of the study were made available to Virginia Tech for use in this project. Similar to the NCHRP 322 project, this project first performed a survey to determine the state-of-the-practice for precast, prestressed girders made continuous. Next, the project investigated the different prediction models and developed a new model to predict the magnitude of continuity moments. Six full scale stub specimens were fabricated and tested to determine the effectiveness of different continuity details. Two full scale and full length specimens were also fabricated and tested. Based on the results of the survey and the testing, recommendations were made for design and detailing of continuity connections.

Many of the results from the survey and the testing program were similar to results found in the earlier NCHRP 322 project. Some of the different results will be presented. The report indicated that the Alabama Department of Transportation (ALDOT) had a number of bridges in its inventory that had experienced damage by cracking in either the continuity diaphragm or in the ends of the girders. Of the approximately 2000 girders, about 74 had experienced cracking. The ends of the girders were detailed to extend three inches into the diaphragm. ALDOT concluded that moments due to thermal gradients caused the problems.

Similar to the NCHRP 322 report, it was concluded that the net positive moment at the midspan of a girder made continuous is essentially independent of the amount of positive moment reinforcing used in the positive moment connection. However, analytically, it was determined that the amount of positive moment reinforcing steel provided above a support can have an effect on the behavior of a continuous system. For the positive moment steel in a continuity connection, as the amount of steel increases, the continuity of the system increases, causing higher negative moments over the supports and lower positive moments at midspan. Also, as the amount increases, the cracking in the bottom of the continuity connection decreases while the positive restraint moment increases.

It was recommended that the amount of positive moment steel in the continuity connection not be greater than that which will provide a moment capacity of 1.2 times the cracking moment. Parametric studies showed that steel levels above this value would not improve the behavior of the continuous systems. The parametric studies also indicated that the continuity of a system that has experienced cracking will never reach 100 percent. Instead, it
was found that continuity over the supports for negative moments was only about 80 percent.

Tests showed that the width of the diaphragm outside the footprint of the girders did not influence the performance of the connection. It was recommended that the diaphragm should be modeled as a T-beam with the web width equal to the bottom flange width of the girder.

From the full-scale full length tests, it was observed that daily temperature changes caused end reactions to change by as much as 20 percent during the observation period. This translates into restraint moments as high as 60 percent of the cracking moment or 2.5 times the anticipated live load plus impact moment. It was concluded that thermal influences should be considered.

It was also observed that there was not as much restraint moment caused by the differential shrinkage as predicted by the models. It was suggested that reinforcing in the deck or wet weather conditions may have influenced the behavior.

The first test performed on the full scale specimens used a diaphragm which was placed in stages. First, concrete was placed in the bottom 1/3 of the diaphragm alone (no concrete was placed for the deck) and allowed to cure for 28 days. Next, concrete for the remaining diaphragm and slab was placed. This sequence was intended to allow the bottom of the diaphragm to become precompressed when the slab is placed due to rotations at the end of the girder. For this specimen, the heat of hydration of the deck when placed caused the top of the composite system to expand, resulting in reduced center support reactions. The system cambered up enough to completely relieve the precompression caused by the slab placement. After hydration of the slab was complete and the slab had cooled, the top of the composite section contracted, causing the center support reactions to increase. After the slab had cooled, it was observed that a net negative moment had been induced in the connection. Therefore, the partial diaphragm had successfully caused precompression in diaphragm, but possibly not because of the rotation of the end of the girders as originally anticipated, but instead due to the thermal movement of the slab.

In the conclusions, it was indicated that the additional costs for providing continuity for live loads was about $200 per girder. The magnitude of the positive moment for design should be limited to 1.2 times the cracking moment. If higher values are required, a minimum age of continuity should be required in the contract documents to prevent such a high moment from
developing. Although the parametric studies indicated that cracking in the continuity diaphragms would reduce the continuity of the system, this reduction was not observed in the full size test specimens except at near ultimate loads. When cracking at the bottom of the connection first occurred, the crack would not immediately extend all the way up into the slab as predicted by the parametric studies.

2.3 Missouri

From 1971 to 1974, the Missouri State Highway Commission in cooperation with the Federal Highway Administration investigated the ability to use extended prestressing strands to develop the positive restraint moment in a continuity connection for prestressed girders. The results of this research were published in a report titled “End Connections of Pretensioned I-Beam Bridges (Salmons 1975).” The research contained in the report was divided into two main topics. First, a study was performed to determine the relationship of the embedment length of untensioned prestressing strands to the strength of the strands. The second study focused on the embedment length and strength relationship of the strands in full scale I-beam continuity connections.

A total of 69 small scale pullout specimens were tested in the first part of the project to help develop the relationship between the untensioned prestressing strand and ultimate pullout strength. Straight strand pullout tests were performed on both unfrayed and frayed strands while bent strand pullout tests were performed on only unfrayed strands. The embedment lengths varied from 4 in. to 45 in. for the straight strands and from 10 in. to 40 in. for the bent strands. The concrete used for the specimens ranged from 3,750 psi to 6,900 psi. Four different strand diameters were used: 3/8 in., 7/16 in., ½ in., and 6/10 in.

The general mechanisms providing the bond of an initially untensioned strand to the concrete were addressed. Since the strands are made of helically wrapped individual wires (seven wires for a typical prestressing strand), the strand tends to unscrew through the concrete as load is applied. By itself, this unscrewing effect would not provide for a good bond to the concrete. However, as the strand is loaded, it also stretches due to the tensile forces. This stretching causes the pitch, or the distance between individual wire threads, to increase. Since the pitch which is formed in the concrete does not change, normal and frictional forces are created when the strands are stretched which are greater in magnitude than the loss in forces due
to the unscrewing effect.

Results of the bond testing indicated that both the concrete strength and the strand diameter had a negligible influence on the ultimate bond strength. Also, the results from specimens containing some confinement reinforcement showed little difference from the results from similar specimens without confining reinforcement.

The relationship between the bond stresses at slip and the embedment length of the strands was developed for each of the three basic cases, straight unfrayed strands, straight frayed strands, and bent unfrayed strands. For each of the cases, the relationship is:

\[ f_s = 6.14L_e - 22.35 \quad \text{for Bent Strands Unfrayed} \]
\[ f_s = 2.97L_e + 1.16 \quad \text{for Straight Strands Unfrayed} \]  \hspace{1cm} (2.5)
\[ f_s = 1.18L_e + 0.79 \quad \text{for Straight Strands Frayed} \]

where \( f_s \) is the slip stress in kips per square inches and \( L_e \) is the embedment length in inches.

For design, the above equations were modified for a lower bound which is intended to give the relationship that will prevent general slip. These equations are:

\[ L_e = 0.163f_s + 8.25 \quad \text{for Bent Strands Unfrayed} \]
\[ L_e = 0.337f_s + 8.00 \quad \text{for Straight Strands Unfrayed} \]  \hspace{1cm} (2.6)
\[ L_e = 0.552f_s + 5.50 \quad \text{for Straight Strands Frayed} \]

where \( f_s \) is the slip stress in kips per square inches and \( L_e \) is the embedment length in inches.

Six full scale I-beam continuity connections were tested for the second phase of the project. The beams were Missouri Type II prestressed, precast I-beams 32 in. deep. The beam specimens were 6 ft-3 in. long with 12 extended \( \frac{1}{2} \) in. diameter prestressing strands bent at 90 degrees. The beams were placed end to end with a 7 in. gap and connected with a 2 ft 6 in. diaphragm. Therefore, the ends of the beams were embedded 11 \( \frac{1}{2} \) in. within the diaphragm. An 8 ft wide by 6 \( \frac{1}{2} \) in. deep deck was cast on top of a 1 in. bolster for three of the tests. The other three tests did not have a deck cast. The specimens were tested in flexure by supporting the beams at each end and applying a load downward at the center to produce a positive moment.

The specimens were first loaded in the elastic range and the response was monitored with deflection and strain gage readings. The specimens were then loaded in the inelastic region to monitor inelastic behavior. Finally, the specimens were loaded to failure to determine the
Two different theories were used to predict the stress-strain behavior of the untensioned prestressing strand. The first was a simple linear elastic stress-strain relationship while the second was a stress-apparent strain relationship developed in earlier work. The ultimate strength of the untensioned strands was taken to be 15 percent greater than that which would be predicted by the relationships developed in the first phase of this study for general slip.

The specimens performed well under the testing and it was concluded that untensioned prestressing strands can be used for the primary reinforcing of a positive moment connection. It was also concluded that at ultimate loads, the tension forces within the diaphragm are greatest at the centerline of the beam and decrease quickly from the center outward. One tie (stirrup) located 3 ½ in. from the edge of the beam did exceed its yield strain at failure while another tie located an additional 12 in. outward did not even reach yield at the time the connection failed.

Based on the results of both phases of the project, recommendations were made for the design of connections made with untensioned prestressing strands. The recommendations were based on lower bound relationships in the testing data and the fact that the point where general slip occurs can be assumed to be a conservative estimate for the ultimate strand force. The relationships were provided for both working stress and ultimate strength analyses. It was recommended that the relationship at general slip be used for the ultimate strength check and that this relationship be multiplied by 1.4 for working stress analysis.

For the ultimate capacity of the untensioned prestressing strand the recommended equations are:

\[
L_c = 0.163f_{pu} + 8.25 \quad \text{for } L_{pb} \leq 8.25
\]

\[
L_c = 0.163 \left[ f_{pu} - \left( \frac{L_{pb} - 8.25}{0.337} \right) \right] + L_{pb} \quad \text{for } L_{pb} > 8.25
\]

(2.7)

where \(L_c\) is the total embedment length of the strand in in., \(L_{pb}\) is the prebend length of the strand in in., and \(f_{pu}\) is the stress in strand at general slip in ksi.

Considering no slip in the strands and that strains are still elastic, a relationship for the stress in the strand at general slip was developed:
Knowing the number of strands and dimensions of the untensioned prestressing strand, the ultimate moment capacity of the connection was developed by ignoring the contribution of any compression steel and including only the diaphragm ties (stirrups) that are within 6 in. of either side of the beam. This moment is given as:

\[ M_u = \phi \left[ A_{ps} f_{pu} \left( d_{ps} - \frac{a}{2} \right) + A_s f_y \left( d - \frac{a}{2} \right) \right] \]  

(2.9)

where \( \Phi \) is 0.9 for flexure, \( A_{ps} \) is the area of the untensioned prestressing strands, \( d_{ps} \) is the distance to the centroid of the prestressing strand from the compression face, \( a \) is the depth of the compression stress block, \( A_s \) is the area of the diaphragm steel within 6 in. of the beam, and \( d \) is the distance to the centroid of the diaphragm steel from the compression face.

2.4 Incremental Analyses

2.4.1 RMCALC

RMCALC is a computer program copyrighted by Michael McDonagh of Entranco, Inc. (McDonagh 2001). The program is available online as part of the Washington State Department of Transportation’s Alternate Route Project, where terms and conditions of its use are made available.

The program is similar to the program BRIDGERM which was developed as part of the NCHRP 322 project. The only difference is that RMCALC is written in Visual Basic while BRIDGERM was written in Fortran. Sample calculations that were performed in both programs showed that the two programs give the same results.

The program determines restraint moments in a continuous girder system due to creep and shrinkage. Thermal effects are not considered in the program. To determine the restraint moments, an incremental time step solution is performed. The program uses ACI-209 creep and shrinkage models published in 1982. Prestress losses are determined based on the PCI Committee on Prestress Losses recommendations published in 1975. The influence of the reinforcing in the deck on the shrinkage of the deck is also considered. Unlike some programs and design procedures, RMCALC considers the actual length of the continuity diaphragm in the
direction of the span as a small interior span with a continuous pinned support at each end of the diaphragm. Special routines are used to determine if the restraint moments which are created would ever produce a situation where the reaction would become upward (or negative) at the end of girder and diaphragm interface.

The code for this program is also available online, and has been compared to the Fortran code used for BRIDGERM. The two appear almost identical, and both are extremely difficult to follow because of the high number of nested GOTO statements. Some specific aspects of the program have been examined more closely than others, and a few of the assumptions made in the program are debatable. The program uses different routines for two and three span structures, and the results for a structure that is similar in all aspects except for the number of spans are difficult to explain.

2.4.2 Flexibility Based Model

Recently in 2001, another analytic tool was developed to determine the effects of the time dependent restraint moments on precast, prestressed girders with cast-in-place decks made continuous (Mirmiran, et al 2001). This tool uses a flexibility based model to consider the effects of creep, shrinkage, prestress losses, age of loading and the sequence of loading on the development of the restraint moments. Prestress losses are estimated using the recommendations of PCI and the correction factors from ACI 209 are used for relative humidity, loading age and volume-to-surface ratio. The model uses incremental time steps, similar to the RMCALC program, but also accounts for the non-linear changes that occur in the stress-strain relationship and the changes in the stiffness of the section along its length. To account for the variation of the stiffness along the member length, the Moment-Curvature relationship is developed for the member along its length. The commonly used program RESPONSE, which was originally developed by Collins and Mitchell, is used in creating the Moment Curvature relationship (Mirmiran, et al 2001). The analytic model was used first to predict the behavior of the specimens tested in the early PCA tests, see Section 2.1.1. Secondly, a parametric study was performed to investigate the effects the development of the restraint moments would have on the overall continuity of the system.

The model was used to calculate the change in the center support reaction for test specimen 3/4 of the 1960’s PCA tests, which had a positive moment continuity connection. The
ultimate creep coefficient was taken as 2.3 and the ultimate shrinkage strain was taken as 600 micro-strain for both the girder and the deck concrete. The diaphragm section was modeled as a T-section with its web width equal to the width of the bottom flange of the girder. The predicted results were compared to three other prediction methods including the Rate of Creep Method and the Effective Modulus Method. It was found that the model did a better job of predicting the change in the support reaction for the time period from the casting of the deck to approximately 150 days after the removal of the deck formwork. During this period, the reaction at the center support increased, indicating that a negative moment was being formed over the support. After approximately 150 days until the end of observation of the test specimen at 680 days, the predicted change in reaction was negative, indicating the creation of a positive moment over the center support. Also during this period, the predicted change in reaction was slightly less than the observed values in the PCA test, but generally in good agreement with the observed results.

The analytic model was also used to predict the behavior of PCA test specimen 1/2 which had no positive moment reinforcement. The model predicted cracking at the bottom of the diaphragm to occur at approximately 200 days with a decrease in support reaction of about 0.2 kips. At this point, the model would assume that the change in the center support reaction would remain constant at this value until the ultimate curvature of the diaphragm is reached. The observed cracking occurred at approximately 330 days at a decrease in support reaction of about 0.6 kips and continued to decrease to approximately 0.75 kips. This indicates that the model predicted a smaller positive moment than actually observed. The authors believe that the use of a steel bearing plate that was placed fully beneath the diaphragm for the testing may have caused friction forces opposing the opening of the bottom of the continuity reaction. These friction forces were not accounted for in the model.

A parametric study was also performed using the same girder originally used in the PCA tests. The study varied the amount of steel used in the positive moment connection as well as the effective width of the diaphragm, the age at continuity and the age at which the live load was established. The ability of the two span structure to allow distribution of moments along the length of the system was investigated. The input parameters were varied and a Continuity Index was calculated for the moments at mid-span and over the center support. The Continuity Index was defined as the ratio of the predicted moment from this non-linear analysis to the moment predicted using elastic analysis of a section with constant moment of inertia. Full elastic
continuity gives an index value of one.

It was found that the continuity index for the negative moment over the support varied for different amounts of positive moment reinforcing used. Using higher amounts of steel for the positive moment connection and requiring later ages of continuity both would produce better continuity indices compared to lower amounts of steel and early ages of continuity. Nonetheless, for all inputs considered, the continuity index for the total moment over the support always remained less than one. This indicated that the predicted moment over the support would always be less than the predicted moment obtained using elastic analysis.

The continuity index for total moments at mid-span was found to always be greater than one, indicating the predicted moments at mid-span would always be greater than the predicted moments using elastic analysis. Although the predicted moments were found to always be greater than the elastic moments, the magnitude of the moments did not vary significantly due to either the age of applied live load or the amount of positive moment reinforcing steel. For smaller amounts of positive moment reinforcing steel, cracking would develop in the bottom of the diaphragm and the restraint moments would not increase from that point significantly. Likewise, the cracking at the support would reduce the stiffness of the system at the support, causing higher mid-span moments to develop due to the live load. For higher amounts of reinforcing steel, the opposite effect would occur. The higher steel would allow for larger restraint moments to be induced over the support, causing a higher restraint moment to occur in mid-span. Also, the higher amount of steel would increase the stiffness of the system at the support, and allow for a reduced positive moment at mid-span. The sum of the restraint moment at mid-span and the live load moment at mid-span was found to be essentially the same for all levels of positive moment reinforcing steel (Mirmiran, et al 2001).

2.5 Prediction of Concrete Response

2.5.1 ACI 209

The American Concrete Institute (ACI) committee 209 has created a unified model to predict the effects of moisture changes, sustained loads, and temperature changes for both reinforced and prestressed concrete (ACI 2002). This model, often referred to as ACI 209, is intended to be used for the prediction of material responses under service conditions. The model gives recommended ways to calculate the compressive strength, modulus of rupture, tensile
strength, modulus of elasticity, creep, shrinkage, and age-adjusted effective modulus over time.

The compressive strength, \(f'_c\), in pounds per square inches (psi) at any time \(t\) is given by the formula:

\[
(f'_c)_t = \frac{t}{\alpha + \beta t} (f'_{c28})
\]

where \(t\) is the age of the concrete in days, \((f'_{c28})\) is the 28-day compressive strength in psi, and \(\alpha\) and \(\beta\) are constants depending on the curing method and cement type. For steam cured concrete, recommended values for \(\alpha\) and \(\beta\) are: 1.0 and 0.95 respectively for Type I cement and 0.7 and 0.98 respectively for Type III cement.

The modulus of rupture, \(f_r\), the direct tensile strength, \(f'_t\), and the modulus of elasticity, \(E_{ct}\), are given by the following formulas:

\[
\begin{align*}
  f_r &= g_r \left[ w (f'_c) \right]^{1/2} \\
  f'_t &= g_t \left[ w (f'_c) \right]^{1/2} \\
  E_{ct} &= g_{ct} \left[ w^3 (f'_c) \right]^{1/2}
\end{align*}
\]

where \(w\) is the unit weight in pounds per cubic foot, \(g_r\) is a constant ranging from 0.6 to 1.0, usually taken between 0.6 and 0.7, \(g_t\) is 1/3, and \(g_{ct}\) is 33.

The creep coefficient, \(\nu_t\), at a given time \(t\), is defined as the ratio of the creep strain to the initial strain. The formula for the creep coefficient at any time \(t\) in days for concrete that is loaded at an age of seven days and undergoes either moist curing or one to three days of steam curing is:

\[
\nu_t = \frac{t^{0.60}}{10 + t^{0.60}} \nu_u
\]

where \(t\) is the time in days after loading takes place and \(\nu_u\) is the ultimate creep coefficient modified by the product of all applicable correction factors. When specific test data is not available, a value of 2.35 is recommended for \(\nu_u\).

The ultimate creep coefficient is multiplied by a correction factor for creep, for example, \(\nu_u=2.35 \gamma_c\). The factor, \(\gamma_c\), is the product of the correction factors for loading age (\(\gamma_{la}\)), ambient relative humidity (\(\gamma_h\)), average thickness or volume to surface ratio (\(\gamma_h\) or \(\gamma_{vs}\)), concrete slump
(γₙ), fine aggregate percentage (γₚ), and air content (γₐ). All correction factors for creep are shown below:

\[
\begin{align*}
\text{Loading Age, } \gamma_{la} & \quad \gamma_{la} = 1.25(t_{la})^{-0.118} & \text{for Moist Cured Concrete} \\
\gamma_{la} & \quad \gamma_{la} = 1.13(t_{la})^{-0.094} & \text{for Steam Cured Concrete} \\
\text{Ambient Humidity, } \gamma_{\lambda} & \quad \gamma_{\lambda} = 1.27 - 0.0067\lambda & \text{for } \lambda > 40 \\
\text{Average Thickness, } \gamma_{h} & \quad \gamma_{h} = 1.14 - 0.023h & \text{for First Year After Loading} \\
\text{Volume – surface Ratio, } \gamma_{vs} & \quad \gamma_{vs} = \left(\frac{2}{3}\right)^{1.13} \exp(-0.54 v/s) \right] \\
\text{Slump, } \gamma_{s} & \quad \gamma_{s} = 0.82 + 0.067s \\
\text{Fine Aggregate Percent, } \gamma_{\psi} & \quad \gamma_{\psi} = 0.88 + 0.0024\psi \\
\text{Air Content, } \gamma_{\alpha} & \quad \gamma_{\alpha} = 0.46 + 0.09\alpha & \text{Greater than 1}
\end{align*}
\]

where \(t_{la}\) is the loading age in days, \(\lambda\) is the relative humidity as a percentage, \(h\) is the average thickness in inches, \(v/s\) is the volume to surface ratio in inches, \(s\) is the slump in inches, \(\psi\) is the percentage of fine aggregate to the total aggregate by weight, and \(\alpha\) is the air content as a percent.

For a concrete having a slump less than 5 in., air content less than 8 percent, a cement content ranging from 470 lbs. per yd\(^3\) to 750 lbs. per yd\(^3\), and a fine aggregate percentage ranging from 40 percent to 60 percent, the product of the corrections for concrete composition is generally about 1.0. Generally, ACI 209 gives an ultimate creep coefficient range of 1.5 to 2.5.

Two different formulas to predict the shrinkage strain at a given time \(t\), \((\varepsilon_{sh})_t\), are provided, one for concrete which undergoes 7 days of moist curing and one for concrete which undergoes 1 to 3 days of steam curing. These formulas are:

\[
\begin{align*}
(\varepsilon_{sh})_t = \frac{t}{35+t}(\varepsilon_{sh})_u & \quad \text{for 7 days Moist Curing} \\
(\varepsilon_{sh})_t = \frac{t}{55+t}(\varepsilon_{sh})_u & \quad \text{for 1–3 days Steam Curing}
\end{align*}
\]

where \(t\) is the time in days after curing and \((\varepsilon_{sh})_u\) is the ultimate shrinkage strain. A recommended value for the ultimate shrinkage strain is 780 x 10\(^{-6}\) in./in. when specific test data is not available.

Similar to the ultimate creep coefficient, the ultimate shrinkage strain is multiplied by a
correction factor for shrinkage, $\gamma_{sh}$. For example, $(e_{sh})_u = 780 \gamma_{sh} \times 10^{-6}$ in./in.. The correction factor for shrinkage, $\gamma_{sh}$, is the product of the correction factors for duration of initial moist cure ($\gamma_{cp}$), ambient relative humidity ($\gamma_{\lambda}$), average thickness or volume to surface ratio ($\gamma_h$ or $\gamma_{vs}$), slump ($\gamma_s$), fine aggregate percentage ($\gamma_\psi$), cement content ($\gamma_c$), and air content ($\gamma_\alpha$). These correction factors are shown below:

\[
\begin{align*}
\text{Duration of Moist Curing, } \gamma_{cp} & \quad \gamma_{cp} = [1.2, 1.1, 1.0, 0.93, 0.86, 0.75] \quad \text{for } [1, 3, 7, 14, 28, 90] \text{ Days} \\
\text{Ambient Humidity, } \gamma_{\lambda} & \quad \gamma_{\lambda} = 1.40 - 0.010\lambda \quad \text{for } 40 \leq \lambda \leq 80 \\
& \quad \gamma_{\lambda} = 3.00 - 0.030\lambda \quad \text{for } 80 > \lambda \leq 100 \\
\text{Average Thickness, } \gamma_h & \quad \gamma_h = 1.23 - 0.038h \quad \text{for First Year of Drying} \\
& \quad \gamma_h = 1.17 - 0.029h \quad \text{for Ultimate Shrinkage} \\
\text{Volume} – \text{surface Ratio, } \gamma_{vs} & \quad \gamma_{vs} = 1.2 \exp(-0.12v/s) \\
\text{Slump, } \gamma_s & \quad \gamma_s = 0.89 + 0.041s \\
\text{Fine Aggregate Percentage, } \gamma_\psi & \quad \gamma_\psi = 0.30 + 0.014\psi \quad \text{for } \psi \leq 50 \text{ percent} \\
& \quad \gamma_\psi = 0.90 + 0.002\psi \quad \text{for } \psi > 50 \text{ percent} \\
\text{Cement Content, } \gamma_c & \quad \gamma_c = 0.75 + 0.00036c \\
\text{Air Content, } \gamma_\alpha & \quad \gamma_\alpha = 0.95 + 0.008\alpha
\end{align*}
\]

where $\lambda$ is the relative humidity as a percentage, $h$ is the average thickness in inches, $v/s$ is the volume to surface ratio in inches, $s$ is the slump in inches, $\psi$ is the percentage of fine aggregate to the total aggregate by weight, $c$ is the cement content in pounds per cubic yard, and $\alpha$ is the air content as a percent.

ACI 209 also provides guidance for indeterminate structures that experience a redistribution of internal forces due to imposed deformations or a change in the statical system. It states in “Chapter 5 – Response of Structures with Significant Time Change of Stress”, that a difference in creep properties which may be present due to differences in the concrete properties or temperature changes may cause a redistribution of forces in an indeterminate structure. These forces are relaxed due to creep in the concrete, which is called aging of the concrete. One method presented to account for the aging of the concrete is to perform an elastic analysis and use an age-adjusted modulus, $E_{ca}$, in place of the modulus of elasticity. This age adjusted modulus is given by:
where $E_{ci}$ is the initial modulus of elasticity of the concrete, $X$ is the aging coefficient, and $\nu_t$ is the creep coefficient at time $t$. The aging coefficient, $X$, depends on the age that the concrete is first loaded, $t_{la}$, and the duration that the load has been applied, $t-t_{la}$. Table 2.3 gives values for the aging coefficient. It is recommended that interpolation in the table be based on log-log interpolation instead of linear interpolation.

### Table 2.3 – Aging Coefficient for ACI 209

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<th>$t-t_{la}$ in days</th>
<th>$t_{la}$ in days</th>
<th>$\nu_u$</th>
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<td></td>
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<td>0.839</td>
<td>0.946</td>
<td>0.951</td>
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<tr>
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<td>0.511</td>
<td>0.912</td>
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<td>0.981</td>
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<td></td>
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<td>0.943</td>
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<td>0.956</td>
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<td>0.983</td>
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<td>0.818</td>
<td>0.958</td>
<td>0.989</td>
<td>0.997</td>
</tr>
</tbody>
</table>

Using this method, ACI-209 provides guidance for determining the effective stress, moment, or force in a statical system due to a change in the statical system and the effect of aging. Given a structural System (1) at a time $t_1$ which is changed to a structural System (2), then the effective structural System at time $t$, $(S)_t$, for a suddenly applied load is given by:

$$
(S)_t = S_1 + (S_2 - S_1) \left[ \frac{\nu_t - (\nu_t)_t}{(1 + X\nu_t)} \right]
$$

and for a progressive change or slowly applied load is given by:
\( (S)_t = S_1 + (S_2 - S_1) \left[ \frac{(v_t)}{1 + X(v_t)_1} \right] \)  

(2.18)

where \( v_t \) is the creep coefficient at time \( t \), and \((v_t)_1\) is the creep coefficient at time \( t_1 \).

### 2.5.2 CEB-FIB Model Code 1990 (MC-90)

The CEB-FIB, *Model Code for Concrete Structures, 1990* (MC-1990), provides predictions for the time effects of temperature, shrinkage, and creep on concrete (Comite 1990). This model code, which was developed in Europe, is based on SI units and will be presented in these units with conversions to English units where appropriate. The code is intended for concretes having characteristic compressive strengths, \( f_{ck} \), which range from 12 Megapascals (MPa) to 80 MPa (1.74 ksi to 11.6 ksi). The characteristic compressive strength is the compressive strength of a cylinder, which is often referred to as the specified compressive strength, \( f'_c \), in the United States. Instead of the characteristic compressive strength, the model uses the mean compressive strength, \( f_{cm} \), which is:

\[
 f_{cm} = f_{ck} + 8 \text{ MPa}
\]  

(2.19)

where 8 MPa is equal to 1.16 ksi. The model is also intended for sustained loads that produce stresses which are less than forty percent of the mean compressive strength, relative humidity from 40 percent to 100 percent, and temperatures from 5°C to 30°C (41°F to 86°F).

The creep strain which develops at time \( t \) from an applied stress at time \( t_0 \) is given as \( \varepsilon_{cc}(t,t_0) \) which is:

\[
 \varepsilon_{cc}(t,t_0) = \frac{\sigma_c(t_0)}{E_{ci}} \phi(t,t_0) 
\]  

(2.20)

where \( \sigma_c(t_0) \) is the applied stress at time \( t_0 \), \( E_{ci} \) is the 28 day modulus of elasticity, and \( \phi(t,t_0) \) is the creep coefficient. The creep coefficient is a product of the notional creep coefficient, \( \phi_o \), and a coefficient \( \beta_c(t-t_0) \). The notional creep coefficient is given by:

\[
 \phi_o = \phi_{RH} \beta(f_{cm}) \beta(t_0)
\]  

(2.21)

where the first term accounting for changes in relative humidity is given by:
\[
\phi_{RH} = 1 + \frac{1 - \frac{RH}{RH_o}}{0.46\left(\frac{h}{h_o}\right)^{1/3}}
\]  

(2.22)

where RH is the relative ambient humidity as a percentage, RH\(_o\) is 100\%, h is the notational size of the member in mm, and h\(_o\) is 100 mm. The notational size is determined by:

\[
h = 2A_c / u
\]

(2.23)

where \(A_c\) is the cross sectional area and \(u\) is the perimeter exposed to the atmosphere. The second term which accounts for the applied compressive strength is:

\[
\beta(f_{cm}) = \frac{5.3}{(f_{cm} / f_{cmo})^{0.5}}
\]

(2.24)

where \(f_{cm}\) is the 28 day mean compressive strength and \(f_{cmo}\) is 10 Mpa (1.45 ksi). The term accounting for the age at loading is:

\[
\beta(t_o) = \frac{1}{0.1 + \left(\frac{t_o}{t_1}\right)^{0.2}}
\]

(2.25)

where \(t_o\) is the age of loading in days and \(t_1\) is one day. The age of loading may be adjusted based on the type of cement used in the concrete and the temperature during curing. To make this adjustment, the adjusted age of the concrete at the time loading takes place, \(t_{o,T}\), is first determined as a function of the number of days the prevailing external temperature is different than 20°C (68°F). This expression is:

\[
t_{o,T} = \sum_{i=1}^{n} \left[ \Delta t_i e^{-\left(\frac{4000}{273 + T(\Delta t_i)/T_o}\right)} \right]
\]

(2.26)

where \(T(\Delta t_i)\) is the external temperature which occurs for \((\Delta t_i)\) days and \(T_o\) is 1°C. It is probably best to convert external temperatures in °F to °C and simply use this equation. Once the adjusted age of loading is obtained, the adjustment for the type of cement is made by:
\[ t_o = t_{o,T} \left[ \frac{9}{2 + (t_{o,T} / t_{1,T})^{1/2}} + 1 \right]^\alpha \geq 0.5 \text{ days} \]  

(2.27)

where \( t_{1,T} \) is one day and \( \alpha \) is a function of the cement type which equal -1.0, 0, or 1.0 for slowly hardening cement, normal or rapidly hardening cement, and rapidly hardening high strength cements, respectively.

The time function for the development of creep is \( \beta_c(t-t_0) \), where \( t \) is the time in days to determine the creep and \( t_0 \) is the previously determined adjusted age at which loading takes place. The time function is given by:

\[ \beta_c(t-t_0) = \left[ \frac{(t-t_0)/t_1}{\beta_H + (t-t_0)/t_1} \right]^{0.3} \]  

(2.28)

where \( t_1 \) is one day and the term \( \beta_H \) accounts for the influence of the ambient relative humidity and is given by:

\[ \beta_H = 150 \left[ 1 + \left( \frac{1.2 \frac{RH}{RH_o}}{h/o} \right)^{18} \right] \frac{h}{h_o} + 250 \leq 1500 \]  

(2.29)

Similar to the formulas to predict creep, the MC-90 uses both a time function, \( \beta_s \), and a notational shrinkage coefficient, \( \epsilon_{cso} \), to predict the effects of shrinkage. The shrinkage which has occurred from a given time \( t_s \) to a time \( t \), both in days, is given by:

\[ \epsilon_{cs}(t, t_s) = \epsilon_{cso} \beta_s(t-t_s) \]  

(2.30)

where \( t_s \) is defined as the age of the concrete when shrinkage begins. Normally, this is taken as the age when curing ends. The notational shrinkage coefficient is given by:

\[ \epsilon_{cso} = \epsilon_s(f_{cm}) \beta_{RH} \]  

(2.31)

where the term accounting for the cement type and the mean compressive strength is given by:

\[ \epsilon_s(f_{cm}) = [160 + 10 \beta_{sc} (9 - f_{cm} / f_{c so})] 0^{-6} \]  

(2.32)

where \( \beta_{sc} \) is equal to 4, 5, or 8 for slowly hardening cement, normal or rapidly hardening cement, and rapidly hardening high strength cements, respectively. The term which accounts for the relative humidity is:
\[ \beta_{RH} = -1.55 \left[ 1 - \left( \frac{RH}{RH_o} \right)^3 \right] \quad \text{for } 40\% \leq RH < 99\% \]  
\[ \beta_{RH} = +0.25 \quad \text{for } RH \geq 99\% \]  

The time function based on the previously defined terms is given by the following:

\[ \beta_s(t - t_s) = \left( \frac{(t - t_s)/t_i}{350(h/h_o)^2 + (t - t_s)/t_i} \right)^{0.5} \]  

### 2.5.3 ACI 209 Modified by Huo

Recently, in a paper published in the *ACI Materials Journal*, Huo proposed four modifications to the ACI 209 Method for use when predicting the creep and shrinkage of High Performance Concrete (HPC) (Huo, et al 2001). It was found that the addition of a correction factor to account for higher compressive strength and the change of the constant in the denominator of the time functions helped to provide better predictions of creep and shrinkage of HPC.

For the prediction of shrinkage, the shrinkage strain, \( \varepsilon_{sh} \), is expressed as:

\[ \varepsilon_{sh} = (\varepsilon_{sh})_u \frac{t}{K_s + t} \]  
\[ K_s = 45 - 2.5 f'_c \]  
\[ \gamma_{st,sh} = 1.20 - 0.05 f'_c \leq 1 \]  

where \( t \) is the time in days after the end of curing, \( f'_c \) is the compressive strength in ksi, \( K_s \) is a term to replace the constant value of 35 used in ACI-209, and \( \gamma_{st,sh} \) is a correction factor for strength that is used along with the product of all other correction factors to determine the ultimate shrinkage strain, \( (\varepsilon_{sh})_u \).

For the prediction of creep, the creep coefficient, \( \nu_t \), is expressed as:

\[ \nu_t = (\nu)_u \frac{t^{0.6}}{K_c + t^{0.6}} \]  
\[ K_c = 12 - 0.5 f'_c \]  
\[ \gamma_{st,cr} = 1.18 - 0.045 f'_c \]  

where \( t \) is the time in days after loading, \( f'_c \) is the compressive strength in ksi, \( K_c \) is a term to
replace the constant value of 10 used in ACI-209, and $\gamma_{st,c}$ is a correction factor for strength that is used along with the product of all other correction factors to determine the ultimate creep coefficient, $\nu_u$.

### 2.5.4 Other Creep and Shrinkage Models

Throughout the years, there have been many models proposed to determine the development of creep and shrinkage over time. Recently, an experimental study performed at Virginia Tech was undertaken to determine which of seven currently used models would best predicted the creep and shrinkage behavior of prestressed concrete used in AASHTO girders for the Virginia Department of Transportation’s project, the Pinners Point Interchange. In addition to the three methods detailed previously in this section, the AASHTO-LRFD, the Gardner GL2000, the Tadros, and the Bazant B3 models were investigated (Townsend 2003).

The concrete investigated for Townsend’s study was a VDOT approved A5 mix with a 28 day specified compressive strength of 8000 psi. This is currently a typical mix design used for precast, prestressed girders in Virginia. Also, this is similar to the mix that was used in this research project. Therefore, many of the results and recommendations from Townsend’s study are applicable to this research.

Townsend concluded that for an accelerated cured concrete that “ACI Modified by Huo is the most accurate predictor of total, creep, and shrinkage strain for the Bayshore HSC mixture loaded to 20.7 MPa (3000 psi).” He also recommended that “… creep and shrinkage models should contain modification factors for compressive strength…” and that since the AASHTO Standard Specifications over predicts prestress losses, that a different model more applicable to high strength concrete should be used for predicting prestress losses.

### 2.5.5 Prestress Loss Models

Just like there are many models to predict creep and shrinkage in concrete, there are many different models used to predict prestress losses. For the parametric study, ultimate lump sum prestress losses of 40.5 ksi have been assumed for the basic calculations. For the methods where the amount of prestress loss for a given time interval was required, the percentage was determined using the guidelines in the AASHTO Standard Specifications. The guidelines presented in one of the original papers on prestress losses and the AASHTO guidelines are
presented in this section

In 1979, a paper presented in *Concrete International* by four authors, Zia, Preston, Scott, and Workman established the basis for many of the current models for predicting prestress losses (Zia, et al 1979). The equations presented were intended for use with prestressed members made of minimum 4000 psi concrete and concrete unit weights of at least 115 pounds per cubic foot. Also, the compressive stress in the precompressed tensile zone should be between 350 psi to 1750 psi. Four separate losses were presented, the sum of which gives the total prestress loss.

The Elastic Shortening Losses, ES, are due to the elastic shortening of the concrete when the prestress load is applied. These losses are given by:

\[
ES = K_{es} E_s \frac{f_{cir}}{E_{ci}}
\]  

(2.37)

where \(K_{es}\) is 1.0 for a pretensioned member, \(E_s\) is the modulus of elasticity of the prestressing strand (usually between 27,000 to 28,500 ksi), \(E_{ci}\) is the initial modulus of elasticity of the concrete at \(f'_{ci}\), and \(f_{cir}\) is the stress in the concrete at the level of the prestressing steel due to the prestressing force and self weight. This value is given by:

\[
f_{cir} = K_{cir} f_{spi} - f_g
\]

(2.38)

where \(K_{cir}\) is 0.9 for a pretensioned member, \(f_{spi}\) is the stress in the concrete at the level of the prestressing steel due to the initial prestressing force, and \(f_g\) is the stress in the concrete at the level of the prestressing steel due to the self-weight of the girder.

The prestress loss due to creep in the concrete, CR, for a prestress member with bonded tendons is given by:

\[
CR = K_{cr} \frac{E_c}{E_c} (f_{cir} - f_{eds})
\]

(2.39)

where \(K_{cr}\) is 2.0 for a pretensioned member, \(E_c\) is the 28 day modulus of elasticity of the concrete, and \(f_{eds}\) is the stress in the concrete at the level of the prestressing steel due to superimposed permanent loads.

The prestress loss due to the shrinkage of the concrete, SH, is given by:

\[

\]
\[ SH = 8.2(10)^{-6} K_{sh} E_s \left( 1 - 0.06 \frac{V}{S} \right) (100 - RH) \]  

(2.40)

where \( K_{sh} \) is 1.0 for pretensioned members, \( V/S \) is the volume to surface area ratio of the concrete, and \( RH \) is the ambient relative humidity.

The fourth loss, which is usually the smallest, the relaxation of the tendons, \( RE \), is given by:

\[ RE = \left[ K_{re} - J (SH + CR + ES) \right] C \]  

(2.41)

where the values depend on the strand type and the prestressing level. For grade 270 low relaxation strand, \( K_{re} = 5000 \), \( J=0.04 \), and \( C=1.00 \) for \( f_{pi}/f_{pu} \) (initial prestressing stress/ ultimate prestressing stress) equal to 0.75.

The AASHTO Standard Specifications have similar provisions for determining the loss in the prestressing steel. (AASHTO 2002) The total prestress loss, \( \Delta f_s \), excluding friction losses, is given by:

\[ \Delta f_s = SH + ES + CR_s + CR_i \]  

(2.42)

where each term represents a separate loss.

The shrinkage loss, \( SH \), for a pretensioned member is given by:

\[ SH = 17000 - 150RH \]  

(2.43)

where \( RH \) is the mean annual average relative humidity in percent (usually around 70 or 75 percent for Virginia).

The elastic shortening loss, \( ES \), for a pretensioned member is given by:

\[ ES = \frac{E_s}{E_{ci}} f_{cir} \]  

(2.44)

where \( E_s \) is the modulus of elasticity of the prestressing steel, \( E_{ci} \) is the modulus of elasticity of the concrete at \( f'_{ci} \), and \( f_{cir} \) is the stress in the concrete at the level of the prestressing steel immediately after transfer due to the initial prestressing forces and the self-weight of the beam. To determine the stress, it is allowed to either estimate the stress level or use 0.69 times the ultimate stress of the prestressing steel.
The prestress loss due to creep of the concrete, \( CR_c \), is given by:

\[
CR_c = 12f_{cr} - 7f_{eds} \tag{2.45}
\]

where \( f_{eds} \) is the stress in the concrete at the level of the prestressing steel due to all dead loads except those that were present when the prestressing force was applied (usually the self-weight).

The prestress loss due to the relaxation of the prestressing steel, \( CR_s \), for a pretensioned member with 250 ksi to 270 ksi low relaxation strands is given by:

\[
CR_s = 5000 - 0.10ES - 0.05(SH + CR_c) \tag{2.46}
\]

2.6 Influences to the Design Process

There are many influences that prevent a better understanding of the behavior of continuous systems. These influences can be divided into two main categories, material influences and design influences. A summary of these issues follows.

2.6.1 Material Influences

It has been estimated that concrete as a material is the second most consumed material on earth, second only to water (Mehta 1986). Even though concrete is so widely used, it is still difficult to predict all of the properties of concrete. Most design procedures assume that concrete is a homogenous material; however, the complex structure of hardened concrete is actually made up of three different phases of materials. The solid aggregate particles are contained within a hydrated cement paste. Between the solid particles and the paste exists a thin transition zone (about 10 to 50 µm thick). Although the transition zone is thin, it has a significant influence on the mechanical properties of the hardened concrete. Also, each of the material phases may contain pores and microcracks which allow material to move within the hardened concrete. This allows changes in the environment such as changes in humidity and temperature to cause properties such as the modulus of elasticity and the compressive strength to change with time.

When placed under a compressive load, concrete undergoes an initial elastic strain. If the load is kept constant over a given time period, then additional strain will occur, which is called the creep strain. Creep strain occurs because the water molecules within the concrete can be either packed tighter together or pushed out of the concrete. The creep strain is made of two different types of creep, the basic creep which occurs because of the water molecules being
packed tighter together and the drying creep which occurs when water is pushed out of the concrete. Most of the times, both of these types of creep are considered together and simply called the basic creep (Mehta 1986). Two commonly used models which are used to predict the effect of creep are the ACI 209 and the CEB MC-90 models (see Section 2.5).

Once the curing process has been completed, hardened concrete will possibly undergo a volume change due to differences in the ambient temperature and/or humidity. Drying shrinkage occurs when the ambient humidity is low enough to cause water molecules within the hardened concrete to migrate to the surface and evaporate into the atmosphere. As these water molecules leave the concrete, the solid particles within the concrete pull closer together, causing a decrease in the volume of the concrete. The ACI-209 and CEB MC-90 models also predict the shrinkage effects over time.

Another type of shrinkage that occurs is thermal shrinkage. Thermal shrinkage occurs when hardened concrete is exposed to cooler temperatures. An average value for the coefficient of thermal expansion/contraction is about 5.5(10^{-6}) per ºF. Values for the coefficient have been observed to range from 3.2(10^{-6}) to 7.0(10^{-6}) per ºF. For reinforced concrete, a value of 6.0(10^{-6}) is generally used (Kosmatka, et al 1988). Mehta indicates that the coefficient of thermal expansion of concrete is directly related to and proportional to the coefficient of thermal expansion of the aggregates, since the aggregates account for such a large percentage of the concrete (Mehta 1986). Table 2.4 shows the average coefficient of thermal expansion of concrete made with various types of aggregates.

Beginning with the mixing phase of the concrete, the ingredients producing concrete begin to undergo an exothermic reaction and release energy. This energy is released in the form of heat and is called the heat of hydration. Two distinct phases of heating generally occur. The first phase occurs within minutes of initial mixing and generates a high amount of heat. However, the heat generated in this phase decreases rapidly and is generally not a concern since the concrete is still fluid at the end of this phase. The second phase is believed to begin when ettringite begins to form and peaks at the final set, or the point where the concrete initially becomes solid. For normal portland cements, it has been observed that about 50 percent of the potential heat of hydration occurs within the first 3 days and about 90 percent of the potential heat of hydration occurs within the first 3 months. (Mehta, 1986)
Table 2.4 – Coefficient of Thermal Expansion of Concrete

<table>
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<th>Rock Type</th>
<th>Coefficient of Thermal Expansion millionths / °F</th>
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</thead>
<tbody>
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<td>Chert</td>
<td>6.6  N/A</td>
</tr>
<tr>
<td>Quartzite</td>
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<tr>
<td>Quartz</td>
<td>6.2  N/A</td>
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<tr>
<td>Sandstone</td>
<td>5.2  6.1</td>
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<tr>
<td>Gravel</td>
<td>N/A  6.1</td>
</tr>
<tr>
<td>Marble</td>
<td>4.6  N/A</td>
</tr>
<tr>
<td>Siliceous Limestone</td>
<td>4.6  N/A</td>
</tr>
<tr>
<td>Blast Furnace Slag</td>
<td>N/A  5.0</td>
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<td>Limestone</td>
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</tr>
</tbody>
</table>


2.6.2 Design Influences

2.6.2.1 Differences in Creep and Shrinkage Models

Choosing which creep and shrinkage model to use in the design can have a significant influence on the predicted restraint moments for girders in a continuous system. Comparisons have been made of design methods using different models. The results of these comparisons are included in the paper in Appendix C.

2.6.2.2 Moment Distribution vs. Flexibility

Once a structural system is made continuous, the designer must determine how effective the system is for redistribution of any loading that then occurs on the continuous system. A continuous system with a constant moment of inertia would be expected to allow the distribution of any additional loads to occur according to an elastic analysis of the continuous system. However, if the system contains spans that have moments of inertia that change over time, or spans that undergo imposed deformations prior to the loading, then the system may not distribute the forces according to a conventional elastic analysis. This reduced ability to redistribute the loads to the continuous system is called a reduction in the system’s continuity, or a reduction in the system’s flexural rigidity.
For a continuous multi-girder system that experiences cracking at a continuity diaphragm due to restraint positive moments, a reduction in the system’s continuity is possible because of the cracking. There seems to be little agreement on the issue of whether or not a reduction in the system’s continuity may occur due to cracking caused by creep and shrinkage restraint moments. However, it is clear that whether or not the system is assumed to act as a fully elastic system will effect the moments predicted from the design process.

Many design procedures assume that any restraint moments will distribute throughout the system as if the system remains elastic. In addition, some methods assume that the diaphragm has no width and model the system as a system with a single support at each pier instead of the actual two supports that exist. Ignoring the width of the diaphragm can produce some interesting results when the redistribution of the restraint moments is considered. For example, it could be argued that differential shrinkage will not occur in the diaphragm section between the bearings if the diaphragm and that closure portion of the deck were cast at the same time. To demonstrate the influence that these assumptions can have on the resulting moments, the following six cases (Cases A through F) consider an idealized three span arrangement, each span 100 ft in length, with 2 ft diaphragms at the interior supports. The distribution of a differential shrinkage moment, taken as 1, can be distributed six different ways as shown in Figure 2.1. Not all of these methods presented are correct. The different methods are shown to help illustrate the fact that there is much confusion in the design process.

For Case A and Case B, the width of the diaphragm has been ignored. For case A, the resulting restraint moment can be found at the interior supports by using moment distribution. For this case, the restraint moment is found to be 1.2 times the applied moment of 1 for the interior supports. This can be verified by the moment distribution method. The value of the restraint moment is then used for the design of the diaphragm. The PCA Method uses this approach. There is no distinction between the moment in the end of the girder and the moment in the diaphragm.

For Case B, the resulting restraint moments have been added to the applied moments. This step is necessary to complete the moment distribution process. Instead of a value of 1.2 over the interior supports, the value is 0.2. This moment could be taken as the actual moment acting on the girder. However, this moment probably should not be considered the moment
acting on the diaphragm. Some references present this moment only, which could cause designers to make errors in the design process.

Methods C through F consider the actual lengths of the diaphragms in the determination of the restraint moments. For Methods C and D, the product of the moment of inertia and modulus of elasticity (EI) of the girder and the diaphragm are considered to be equal. For Methods E and F, the EI of the diaphragm is taken as 0.5 times EI of the girder. This ratio is based on the actual moduli of elasticity for 8 and 4 ksi concrete for the girder and diaphragm respectively and a ratio of the cracking moments for the sections based on the different concrete strengths. Also using moment distribution, the resulting restraint and total moments for Cases C
through F are determined.

The resulting moments based on these assumptions are plotted for all six cases and are shown in Figure 2.2. When the final step of the moment distribution method is performed for all cases except Case A, a value is obtained for the moment at the exterior supports. Although there is curvature present at these locations, there is not any applied moment, only self equilibrating moments.

As can be seen from the plot of the moments for the six cases, the magnitude of the moment in the diaphragm can vary from about 0.2 to about 1.5 depending on the assumptions used in the analysis.

In *Deformation of Concrete Structures*, Branson develops and provides equations for determining the moments and deflections in equal span continuous beams for one to five equal spans due to differential shrinkage and creep (Branson 1977). In his derivation, he describes one method of determining the effect of the differential shrinkage and creep is to first remove the redundant interior supports so that the structure is determinate and to apply the differential shrinkage and creep moment, $M_{DS}$, at each end. The resulting moment diagram, which will be uniform moment, gives the “primary” moments. The deflections caused by the “primary”
moments are then obtained using conventional structural analysis. Then, the reactions required to return the beam to zero deflection at locations where the supports were removed are calculated. These reactions produce the “secondary” moments. Finally, the primary and secondary moments are added up to obtain the “total” moments. As presented, for a two span continuous system, the maximum positive moment over the outside supports is given as \( M = M_{DS} \), while the maximum negative moment over the interior support is given by \( M = -\frac{M_{DS}}{2} \).

Branson also shows that the same result can be obtained using a modified form of the moment distribution method. Tables are presented which show formulas for deflections and “total” moments at the supports. It is not specifically stated whether or not these “total” moments should be used for the design of a continuity connection.

One of the other difficulties in determining the actual continuity of this type of continuous system is the fact that it is often difficult to predict the end rotations of girders made continuous. Long term monitoring of continuous multi-girder bridges over time have indicated that once the deck and continuity connection have been established, very little vertical movement tends to occur at the midspan of the structure due to static loads. This movement is the summation of dead load deflection and camber (Castrodale 2002). Since this movement is required to produce the restraining moments and the resulting end rotations, it is possible that these deformations may not occur as predicted.

Since many believe that predicting the actual value of the restraining moments is difficult and unreliable, the positive moment capacity requirement is often set at a minimum of 1.2 times the positive moment cracking moment. This capacity is believed to help distribute any cracking over a sufficient length of the structure so that the width of individual cracks is insignificant. However, it has been suggested that if cracking were to occur, that realistically there may be only three cracks available to help distribute any imposed rotations due to the creep and shrinkage (Wollmann, et al 1999). Therefore, enough reinforcing should be provided to limit the end rotations enough to prevent excessive cracking and yielding of the steel.

2.6.2.3 Thermal Effects

In 1978, Priestley published a paper addressing the influence of temperature gradients on the behavior and design of concrete bridges (Priestley 1978). He indicated that the design for prestressed concrete bridges for temperature effects generally consists of designing the joints and
bearings to accommodate longitudinal movements only due to temperature changes. However, he also indicated that the temperature gradients can produce significant flexural stresses which are often not considered in the design. For a structure made continuous, these flexural stresses can at times be as high as the stresses produced from the live loads.

Although Priestley recommended that stresses induced from temperature gradients should be considered in the design of continuous structures, the manner in which they should be considered is somewhat unclear. At service conditions, he contends that changes in the structure’s flexural rigidity may often reduce the effects of the thermally induced continuity stresses or allow the cracking caused by the thermally induced stresses to spread out over a sufficient length so that the stresses are insignificant.

Considering a three span prestressed bridge made continuous, the moments due to live load at the center of the center span can be positive. Also, the thermally induced moments due to the temperature gradient are expected to be positive and of constant value over the center span. When the combination of the dead loads, live loads, and thermally induced loads causes moments greater than the allowable positive cracking moment at the center of the span, then cracking can be expected. However, since the thermally induced moments are constant value, the change in moments over the center span is gradual, causing the expected cracking due to these moments to extend over a significant length. This cracking can cause a reduction in the flexural rigidity of the structure, which acts to reduce the magnitude of the continuity moments. He states that the prestressing steel, which is located near the bottom at the center of the center span, will also help to minimize cracking.

A different scenario is described where the maximum combination of dead loads and thermally induced moments cause positive moments to occur over an interior support. This case is generally worst for a two span continuous structure without the application of live load. This is also the case that predicts the worst case positive moments over an interior support, which is the goal of this project. He states that for this case, that the application of the “sagging moment”, or positive moment, over the interior support can be expected to cause cracking to occur over a fairly limited zone. Even though the cracking occurs over a short zone, he contends that the flexural rigidity of the structure will not be significantly reduced as long as the cracks do not approach unacceptable widths. In addition, he recommends that mild steel be placed to help
control these cracks since the tendon profile at the ends is generally higher than at the center.

Priestley states that thermally induced actions can be considered as equivalent forces or moments at service conditions. However, he indicates that it is incorrect to consider the thermally induced actions as forces or moments at ultimate conditions. He explains this by showing the difference that a thermally induced deformation has for unfactored service loads and for factored ultimate loads. For a given Force versus Deformation relationship, at service conditions, the Thermal Deformations can be expected to produce equivalent forces since all forces remain within the linear region of the relationship. The factored Thermal Deformations at Ultimate are located within the nonlinear region of the relationship. Therefore, the force produced by these deformations is smaller than the force would be if the deformation occurred within the linear region. Figure 2.3, which is adapted from Priestley’s paper, shows this relationship for an assumed force and deformation relationship and current load factors.

![Figure 2.3 – Force/Deformation Relationship](image)

Although most currently used design methods for continuous girders in multi-girder bridges do not consider thermal effects, it has been found that thermal effects can have a significant influence on the design process. In fact, recent full scale experiments performed for the NCHRP 519 project have indicated that the thermal effects can be as significant as the live load effects (Miller, et al 2004). Results from this study, have indicated that the daily temperature changes can cause end reactions to vary by as much as 20 percent per day. Taking
these changes in end reactions and multiplying by the span length provides a significant restraint moment due to the temperature change. Also, it was observed that after the deck slabs reached final set, the slabs would then cool and contract, which would create a negative moment over the continuity connection. This negative moment is generally not accounted for in design.

The commentary to the LRFD Specification states that “…open girder construction and multiple steel box girders have traditionally, but perhaps not necessarily correctly, been designed without consideration of temperature gradient…” (AASHTO, 2001). To consider the effects of temperature gradient, guidelines are provided in the “AASHTO Guide Specifications – Thermal Effects in Concrete Bridge Superstructures – 1989” (AASHTO, 1989). These guidelines divide the United States into four Maximum Solar Radiation Zones. Virginia is located in zone 3. The temperature differentials for a concrete superstructure are provided for both a positive and a negative temperature gradient. Figure 2.4 shows the positive and negative temperature gradients proposed by AASHTO.

![Positive Temperature Gradient](image1)

![Negative Temperature Gradient](image2)

Figure 2.4 – Positive and Negative Temperature Gradients

where, for a plain concrete deck (no asphalt) in Virginia, the temperatures T1, T2, and T3 are 41 °F, 11 °F, and 4 °F respectively for the Positive Temperature Gradient and T1, T2, T3, and T4 are 11 °F, 6 °F, 2 °F, and 8 °F for the Negative Temperature Gradient.

In order to determine the effects of a temperature gradient, the structure must first be
made determinate by removing a sufficient number of internal redundancies. After the internal redundancies are removed, the self-equilibrating stresses (or forces) are determined. The redundancies are then reapplied, producing the continuity stresses (or forces).

If a structure were exposed to a temperature gradient such as shown in Figure 2.4, and if the individual fibers were allowed to expand or contract independently from each other, then the structure would undergo a strain profile that matched the temperature gradient profile. However, since it is assumed that plane sections must remain plane, the structure cannot undergo this type of movement. Instead, stresses will be developed in the structure which will create a linear strain distribution. These stresses, called the self-equilibrating stresses, can be found by first assuming that the structure is fully restrained and determining the stresses due to the temperature gradient. Next, the axial force and moment required to make the structure fully restrained are calculated. Finally, the summation of the stresses for the fully restrained section and the opposite of the stresses due to the restraining force and moment give the self-equilibrating stresses.

First, the stresses created due to the temperature gradient are calculated assuming that the structure is totally restrained. These longitudinal stresses, \( \sigma_t(Y) \), are determined at a distance \( Y \) from the center of gravity and are given as:

\[
\sigma_t(Y) = E \alpha T(Y)
\]  
(2.47)

where \( E \) is the modulus of elasticity, \( \alpha \) is the coefficient of thermal expansion, and \( T(Y) \) is the temperature at the given distance \( Y \). Next, the restraining axial force, \( P \), is determined by integrating over the depth of the structure:

\[
P = \int_Y \sigma_t(Y)b(Y)dY
\]  
(2.48)

where \( b(Y) \) is the section width at location \( Y \). Likewise, the restraining moment, \( M \), is determined by integrating the product of the stress, the width, and the distance from the centroid over the height of the structure. It can be determined by:

\[
M = \int_Y \sigma_t(Y)b(Y)YdY
\]  
(2.49)

The self equilibrating stresses, \( \sigma(Y) \), are then determined by:
\[
\sigma(Y) = \sigma_i(Y) - \frac{P}{A} \frac{MY}{I}
\]  
(2.50)

where \(A\) is the area of the section and \(I\) is the moment of inertia of the section. Any redundancies that were removed to make the structure determinate are then reapplied. The self-equilibrating stresses, \(\sigma(Y)\), or self-equilibrating forces (P and M), are then redistributed to produce the continuity stresses or forces.

### 2.6.2.4 Force Distribution in Composite Systems

When a structural system is composed of two members joined rigidly together to produce one composite member, any differences in the material properties of the two members may allow for forces to be generated due to restraining forces at the interface of the two members made composite. Differential shrinkage is one type of change that creates internal forces that occur in a composite structural system whenever the net shrinkage and creep strains at the bottom of the deck differ from the net shrinkage and creep strains at the top of the girder. For most prestressed, precast composite girder and cast-in-place slab systems, more net shrinkage and creep will occur at the bottom of the deck than at the top of the girder because some of the creep and shrinkage of the girder has already taken place prior to the time the deck is cast. This net difference will generally cause forces to be created in the composite system, resulting in a net downward deflection.

There are several different methods used to predict the forces created by the differential shrinkage. In his book *Deformations in Concrete Structures*, Branson presents two of the most commonly used elastic-theory methods, the “Composite Section Method” and the “Separate Section Method” (Branson 1977). Both of these methods will produce the same results as long as the system is assumed to act as a linear elastic system.

The Composite Section Method first requires the net differential shrinkage between the deck and girder to be determined. Assuming that the deck will have more net creep and shrinkage than the girder, the net force produced, \(Q\), can be determined by:

\[
Q = DA_1E_1
\]  
(2.51)

where \(D\) is a shrinkage coefficient (essentially the differential shrinkage strain), \(A_1\) is the area of the slab, and \(E_1\) is the modulus of elasticity of the slab. This force, which will cause tension in the slab, is first assumed to act at the centroid of the slab alone. Then, the force is applied to the
composite section, also at the centroid of the slab. Since it is applied at the same location in opposite directions, the net force to the section is zero; however, stresses are still created in the section. Figure 2.5 shows how these stresses can be determined using this method. The combined system in Figure 2.5 is equivalent to the combined system in Figure 2.6.

Assuming that the neutral axis of the composite section is within the girder as shown, and taking tensile stresses to be positive and compressive stresses negative, then the stresses in the top and bottom of the deck, \( \sigma_{1T} \) and \( \sigma_{1B} \), resulting from the differential shrinkage as determined on the equivalent system can be expressed by:
Likewise, the stresses in the top and bottom of the girder, $\sigma_{2T}$ and $\sigma_{2B}$, resulting from the differential shrinkage as determined on the equivalent system can be expressed by:

$$
\sigma_{2T} = \frac{Q}{A_c} - \frac{QYcsYcl}{Ic} \\
\sigma_{2B} = \frac{Q}{A_c} + \frac{QYcsYcb}{Ic}
$$

(2.53)

where $A_1$ and $E_1$ are the area and the modulus of elasticity of the deck and $A_2$ and $E_2$ are the area and the modulus of elasticity of the girder.

The Separate Section Method considers the net force due to differential shrinkage in a different way. Since there will be no net applied force to a composite system due to the differential shrinkage, this method assumes that the forces at the interface of the slab and girder to be equal, opposite, and collinear. The forces, $F_1$ and $F_2$, are assumed to be equal in magnitude to a force $F$. This force $F$ is then assumed to act at the centroids of the deck and girder respectively on the composite section. Each force creates a moment at the centroid of its section; however, since the forces are acting on a composite section, the moment is not simply the force times the moment arm. Instead, the total moment produced by each force must be distributed to the composite section based on a ratio of the individual sections modulus of elasticity times the moment of inertia, $EI$, to the summation of the $EI$ for both sections. This is different than some methods which distribute the moment based on the ratio of the individual sections $EI$ to the $EI$ of the composite section.

Expressions for the strain at the bottom of the deck and the top of the girder are developed, the summation of which is equal to the differential strain. This differential strain is the same as the coefficient $D$ used in the Composite Section Method. An expression for the force $F$ can then be determined by simultaneously solving the equations. The stresses at the top and the bottom of the deck and girder can then be derived using the force $F$ and the moments $M_1$ and $M_2$ which act on the individual deck and girder sections. It is interesting to note that this
method, which determines the stresses in a composite section, does not require a direct
determination of the composite moment of inertia or the location of the composite neutral axis.

Using the same notation shown above for the Composite Section Method, the force \( F \) can
be given by:

\[
F = \frac{E_1 D}{1 + \frac{1}{A_1} \left( \frac{y_1 b + y_2 t}{A_2(E_2/E_1)} \right)^2 + \frac{1}{I_1 + I_2(E_2/E_1)}}
\]

\[
F = \frac{E_2 D}{1 + \frac{1}{A_2} \left( \frac{y_1 b + y_2 t}{A_1(E_1/E_2)} \right)^2 + \frac{1}{I_2 + I_1(E_1/E_2)}}
\]

(2.54)

where \( A \) is the area of the section, \( E \) is the modulus of elasticity of the section and \( D \) is the
differential shrinkage strain coefficient. The subscript 1 indicates the slab section and the
subscript 2 indicates the girder section.

The moments distributed to the slab and girder sections, \( M_1 \) and \( M_2 \), respectively, are:

\[
M_1 = F(y_{1b} + y_{2t}) \frac{E_1 I_1}{E_1 I_1 + E_2 I_2}
\]

\[
M_2 = F(y_{1b} + y_{2t}) \frac{E_2 I_2}{E_1 I_1 + E_2 I_2}
\]

(2.55)

The stresses at the top and bottom of the slab are given by:

\[
\sigma_{1T} = \frac{F}{A_1} - \frac{M_1 y_{1t}}{I_1}
\]

\[
\sigma_{1B} = \frac{F}{A_1} + \frac{M_1 y_{1b}}{I_1}
\]

(2.56)

and the stresses at the top and bottom of the girder are given by:

\[
\sigma_{2T} = -\frac{F}{A_2} - \frac{M_2 y_{2t}}{I_2}
\]

\[
\sigma_{2B} = -\frac{F}{A_2} - \frac{M_2 y_{2b}}{I_2}
\]

(2.57)

In his book, Branson also gives recommended values for the ultimate differential
shrinkage and creep strain for various structural systems. For a prestressed beam made
composite with a cast-in-place deck that is placed while shored, he recommends ultimate shrinkage coefficients, $D_u$, for three time periods between casting of the beam and casting the deck. For 1-month, 2-month, and 3-month periods respectively, he recommends $D_u$ equal to 310, 415, and 470 microstrain (or $10^{-6}$ in./in.). These values represent the effect of differential shrinkage only. He reasons that in shored construction the neutral axis of the composite section is close to the interface of the slab and girder. Therefore, very small forces will be generated by the additional creep of the composite section and these additional creep effects can be neglected.

For a prestressed beam made composite with a cast-in-place deck that is placed while unshored, the interface between the deck and the girder is initially unstressed. As the girder creeps due to the weight of the slab dead load, he claims that the interface will experience stresses that will “… tend(s) to reduce the differential shrinkage and creep effect in concrete-concrete composite beams.” Likewise, he provides recommended values for the ultimate shrinkage and creep coefficient, $D_u$, for the 1-month, 2-month, and 3-month periods respectively equal to 270, 375, and 435 microstrains. These values are lower than those recommended for the shored system. In a footnote, he recommends that the values for the 2-month time period be used in routine calculations.

ACI-209 recommends that another method to account for the effects of aging can be used to model the response of a structure that changes over time. This method was originally proposed by Trost and is called the age-adjusted modulus method, or sometimes the Trost’s Method. In his book *Prestressed Concrete Bridges*, Menn shows how the Trost’s method was developed (Menn 1986). The key to this method is the fact that Trost found that the aging function, $\mu(t,\tau)$, which changes at time $t$ due to a stress applied at time $\tau$, does not vary much with time after the first application of stress. Trost proposed that the function can be considered constant over time, which greatly simplifies the equations. The aging function, $\mu$, can then be considered a function only of the creep coefficient, $\phi_n$, and the correction for the age of concrete at time of loading, $k(\tau_0)$. The aging coefficient may be interpolated from Table 2.5.

### Table 2.5 – Trost’s Proposed Aging Coefficient

<table>
<thead>
<tr>
<th>$\phi_n$</th>
<th>$k(\tau_0)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>0.75</td>
</tr>
<tr>
<td>3</td>
<td>0.85</td>
</tr>
<tr>
<td>1.5</td>
<td>0.8</td>
</tr>
</tbody>
</table>
Therefore, using the above aging coefficient, μ, the strain in the concrete at time t, \( \varepsilon_c(t) \), is given by:

\[
\varepsilon_c(t) = \frac{\sigma_o}{E_c(\tau_o)} \left[ 1 + \phi(t, \tau_o) \right] + \frac{\sigma(t) - \sigma_o}{E_c(\tau_o)} \left[ 1 + \mu \phi(t, \tau_o) \right] + \varepsilon_{cs}(t)
\]  (2.58)

where \( \sigma_o \) is a constant stress applied at time \( \tau_o \), \( E_c(\tau_o) \) is the modulus of elasticity at time \( \tau_o \), \( \phi(t, \tau_o) \) is the creep coefficient for the time period from \( \tau_o \) to t, \( \sigma(t) \) is the stress at time t, and \( \varepsilon_{cs} \) is the shrinkage in the concrete at time t.

Using the fact that the change in curvature in the deck must equal the change in curvature in the girder, a set of simultaneous equations can be established to solve for the ultimate change in strain of a composite section using the Trost’s Method. This method has been used as one of the methods to predict the restraint moments for the PCBT girder and slab system.

The PCA Method defines the moment due to differential shrinkage, \( M_s \), as:

\[
M_s = \varepsilon_s E_b A_b \left( e'_b + \frac{t}{2} \right)
\]  (2.59)

where \( \varepsilon_s \) is the differential shrinkage strain, \( E_b \) is the modulus of elasticity for the deck slab, \( A_b \) is the area of the deck slab, and the quantity \( (e'_b + t/2) \) is the distance from the center of the deck to the centroid of the composite section (Freyermuth 1969). Since the moment is slowly imposed, it is reduced by the factor \( (1 - e^{-\phi})/\phi \). See Section 2.1.2 PCA Method for further discussion on the shrinkage restraint moment.

2.6.2.5 Standard Specifications versus LRFD Specifications

In the United States, there are two design specifications commonly used for the design of highway bridges, the Standard Specifications for Highway Bridges and the LRFD Bridge Design Specifications. The Standard Specifications have been used for many years, and are currently being used by VDOT for bridges designed for Virginia. The LRFD Specifications is a newer document which has not yet been adopted by the commonwealth. VDOT currently plans on adopting the LRFD Specification in October 2007 (Volgyi 2003). The two specifications consider the effects of temperature in different ways, which can influence the design of temperature induced restraint moments.

The organizing of standards related to bridge design in the United States began as early
as 1921. In 1931, theses standards were published as the Standard Specifications, and have been revised and republished 16 times since then. Since the Federal Highway Administration in conjunction with the state departments of transportation have “set a goal” that the LRFD Specifications shall be used for all new design beginning in 2007, the Standard Specifications for Highway Bridges – 17th Edition – 2002, is intended to be the last edition of the Standard Specifications published (AASHTO 2002). Any future changes to this edition will be made as revisions. The goal of the Standard Specifications has been to provide “… minimum requirements …” that shall apply to ordinary bridges less than 500 feet. For concrete design, the specifications allow the designer to choose either a Service Load Design Method, also called Working Stress or Allowable stress, or a Strength Design Method.

General guidelines are given in the Standard Specifications for the design of simple span girders made continuous in Section 9.7.2 “Bridges Composed of Simple-Span Precast Prestressed Girders Made Continuous”. These provisions are as follows:

Section 9.7.2.1 General

“When structural continuity is assumed in calculating live loads plus impact and composite dead load moments, the effects of creep and shrinkage shall be considered in the design of bridges incorporating simple span precast, prestressed girders and deck slabs continuous over two or more spans”.

Section 9.7.2.2 Positive Moment Connections at Piers

9.7.2.2.1 “Provisions shall be made in the design for the positive moments that may develop in the negative moment region due to the combined effects of creep and shrinkage in the girders and deck slab, and due to the effects of live load plus impact in remote spans. Shrinkage and elastic shortening of the pier shall be considered when significant”.

9.7.2.2.2 “Non-prestressed positive moment connection reinforcement at piers may be designed at a working stress of 0.6 times the yield strength but not to exceed 36 ksi”.

58
Section 9.7.2.3 Negative Moments

9.7.2.3.1 “Negative moment reinforcement shall be proportioned by strength design with load factors in accordance with Article 9.14”.

9.7.2.3.2 “The ultimate negative resisting moment shall be calculated using the compressive strength of the girder concrete regardless of the strength of the diaphragm concrete”.

General guidance is also given in Section 3 of the specifications which is the section on Loads.

Section 3.16 Thermal Forces

“Provisions shall be made for stresses or movements resulting from variations in temperature. The rise and fall in temperature shall be fixed for the locality in which the structure is to be constructed and shall be computed from an assumed temperature at the time of erection. Due consideration shall be given to the lag between air temperature and the interior temperature of massive concrete”.

This section also provides recommendations for the temperature rise and temperature fall. These recommendations are 30°F and 40°F respectively for concrete structures in moderate climates and 35°F and 45°F respectively for concrete structures in cold climates.

In Part B of Section 3, provisions are given for the combination of loads. A full table showing all combinations is provided for both the service load combinations as well as the load factor design combinations. These load combinations can be reduced by eliminating any loads that will not exist for the design of bridges composed of precast, prestressed girders made continuous. Considering only dead loads, D, live loads plus impact for the HS-20 loading, L and I, and temperature forces, T, the three load combinations that include temperature are as follows for both the service load design and the load factor design:
For Service Load Design

\[
\begin{align*}
\text{Group (IV)} &= 1.0 \ (D) + 1.0 \ (L+I) + 1.0 \ (T) \quad \% \text{ Basic Unit Stress} = 125 \\
\text{Group (V)} &= 1.0 \ (D) + 1.0 \ (T) \quad \% \text{ Basic Unit Stress} = 140 \\
\text{Group (VI)} &= 1.0 \ (D) + 1.0 \ (L+I) + 1.0 \ (T) \quad \% \text{ Basic Unit Stress} = 140
\end{align*}
\]

(2.60)

For Strength Design

\[
\begin{align*}
\text{Group (IV)} &= 1.3 \ [1.0 \ (D) + 1.0 \ (L+I) + 1.0 \ (T)] \\
\text{Group (V)} &= 1.25 \ [1.0 \ (D) + 1.0 \ (T)] \\
\text{Group (VI)} &= 1.25 \ [1.0 \ (D) + 1.0 \ (L+I) + 1.0 \ (T)]
\end{align*}
\]

(2.61)

For Service Load Design, the % Basic Unit Stress provided is generally considered an amount that the basic allowable stress may be increased to obtain the maximum allowable stress for the combination. However, since section 9.13.1.2 states, concerning prestressed concrete, that “Design shall be based on strength (Load Factor Design) and on behavior at service conditions (Allowable Stress Design) at all load stages that may be critical during the life of the structure from the time prestressing is first applied”, then the % Basic Unit Stress may not be applicable for prestressed design. Also, its applicability to reinforced concrete design, for example the continuity diaphragm, is debatable. Some designers would allow for its use checking the strength while others would not, arguing that since many other loads, such as wind loads, do not contribute to the load combination, then the allowable stresses for the combination should not be allowed to be increased, at least not increased as much as shown.

In an attempt to better consider the variability that occurs in both the response of structural elements and the actions caused by the loads, engineers began to develop a design method that could account for this variability in a more direct way than in the Allowable Stress and Load Factor Design methods. In 1987, a recommendation was made that a new bridge design specification based on Load and Resistance Factor Design (LRFD) should be created. This recommendation lead to the creation of a bridge design specification based on LRFD, the most current being the AASHTO LRFD Design Specifications-U.S. Units – 2001 Interim Revisions (AASHTO 2001). The specifications are considered by many to be the specifications
which will be used by all bridge designers in the near future. Based on statistical methods to better describe variability in actions and responses, the method is intended to provide for designs which lead to levels of safety that are more consistent and for structures that exhibit better serviceability.

Unlike the Standard Specifications which consider load combinations, the LRFD Specifications require that all members satisfy 11 distinct limit states. In addition to the load factor and resistance factor, which are both used in Load Factor design, the LRFD also includes three factors which account for ductility, redundancy and operational importance. Limit states at service conditions, fatigue conditions, fracture conditions, strength conditions, and extreme event conditions are all considered.

In Section 3 – Loads and Load Factors, individual loads and forces are given, some of which do not appear in the Standard Specifications. The loads and forces applicable to precast, prestressed girders made continuous may include: creep (CR), shrinkage (SH), temperature gradient (TG), uniform temperature (TU), vehicular live load (LL), and vehicular dynamic load allowance (IM). These loads are considered along with permanent loads for dead load of structural components and nonstructural attachments (DC), dead load of wearing surfaces and utilities (DW), and accumulated locked in force effects resulting from the construction process (EL). The factored load effect for one limit state, the STRENGTH-I limit state, including only the above loads and forces is given by:

\[ Q_i = \eta_i \left[ \gamma_p (DC + DW + EL) + 1.75(LL + IM) + (0.5 \text{ or } 1.2)(TU + CR + SH) + Y_{TG}(TG) \right] \]  

where \( \eta_i \) is a load modifier based on ductility, redundancy, and operational importance, \( \gamma_p \) is the load factor for permanent loads, and \( Y_{TG} \) is the load factor for the temperature gradient. The load factors for permanent loads and temperature gradient are given both maximum and minimum values, both of which are to be used in whatever combination necessary to produce the maximum positive and negative extremes.

In section 3.4.1, Load Factors and Load Combinations, it states that:
“The load factor for temperature gradient, \( Y_{TG} \), and settlement, \( Y_{SE} \), should be considered on a project-specific basis. In lieu of project-specific information to the contrary, \( Y_{TG} \) may be taken as:

0.0 at the strength and extreme limit states

1.0 at the service limit state when live load is not considered, and

0.50 at the service limit state when live load is considered.”

It is also interesting to note that in the commentary next to this section, it states:

“Open girder construction and multiple steel box girders have traditionally, but perhaps not necessarily correctly, been designed without consideration of temperature gradient, i.e., \( Y_{TG}=0.0 \).”

Similar to the Standard Specifications, the LRFD Specifications also has a general statement addressing superimposed deformations. In Section 3.12, deformations due to uniform temperature, temperature gradient, shrinkage, creep, and settlement are all addressed. Several of the sections that are relevant are:

3.12.1 General

“Internal force effects in a component due to creep and shrinkage shall be considered. The effect of temperature gradient should be included where appropriate. Force effects resulting from resisting component deformation, displacement of points of load application, and support movements shall be included in the analysis.”

3.12.4 Differential Shrinkage

“Where appropriate, differential shrinkage strains between concretes of different age and composition, and between concrete and steel or wood, shall be determined in accordance with the provisions of Section 5.”
3.12.5 Creep

“Creep strains for concrete and wood shall be in accordance with the provisions of Section 5 and Section 8, respectively. In determining force effects and deformations due to creep, dependence on time and changes in compressive stresses shall be taken into account.”

Section 5, Concrete Structures, has guidance on predicting the effects of shrinkage and creep on both conventionally reinforced and prestressed concrete. In Section 5.4.2.3, Shrinkage and Creep, it states that either the provisions in Section 5 or two other methods may be used to estimate shrinkage and creep when specific data is not available. The other two methods it allows are the CEB-FIP model code and the ACI 209 code, see Section 2.5 for these two methods. It also states in the commentary that:

“Without specific physical tests or prior experience with the materials, the use of the empirical methods referenced in these Specifications cannot be expected to yield results with errors less than ± 50 percent.”

Section 3.12.2, Uniform Temperature, provides temperature ranges from 10°F to 80°F for concrete structures in moderate climates and temperature ranges from 0°F to 80°F for concrete structures in cold climates. In the commentary, it defines a moderate climate as one in which the number of freezing days is less than 14. A freezing day is defined as a day in which the average temperature is less than 32°F.

Section 3.12.3, Temperature Gradient, gives design guidelines for determining the internal stresses and or deformations due to the variation of the temperature through an element. This section is similar to the design provisions of the AASHTO Guide Specifications – Thermal Effects in Concrete Bridge Structures – 1989, with some differences. Although the statement in Section 3.12.1, General, states “…the effect of temperature gradient should be included where appropriate…,” it is not clear whether it is appropriate to consider these effects in a composite slab girder system when the girder is a prestressed I-shaped girders constituting a multibeam bridge. In the commentary it states:
“Temperature gradient is included in various load combinations in Table 3.4.1-1. This does not mean that it need be investigated for all types of structures. If experience has shown that neglecting temperature gradient in the design of a given type of structure has not lead to structural distress, the Owner may choose to exclude temperature gradient. Multibeam bridges are an example of a type of structure for which judgment and past experience should be considered.”

2.6.2.6 Aging or Relaxation of Concrete

It has been shown that concrete will respond differently to loads that are applied instantaneously than to loads that are applied slowly over time. Loads such as the differential shrinkage moments and the moments due to prestressing steel losses generally increase from zero to their maximum value over many years. It has been found that since these loads are applied slowly, the creep effects of the concrete are generally less than the creep effects would be if the loads were applied instantaneously. One reason that the response is different is the fact that the modulus of elasticity of the concrete changes over time, generally increasing with age. To account for these slowly applied loads, several methods have been proposed to account for the reduction in the creep effect, which is also called the relaxation or aging effects of the concrete.

Branson provides both a General Solution and a Recommended Approximate Solution for Practice (Branson 1977). The General Solution modifies the Effective Modulus Method to account for relaxation of the concrete. The Recommended Approximate Solution for Practice gives recommended values that can be used for typical conditions and routine calculations.

The General Solution gives recommended values of the modulus of elasticity of the slab, \( E_1 \), the modulus of elasticity of the girder, \( E_2 \), and the final force due to differential shrinkage, \( F_{DS} \). These recommended values are:

\[
E_1 = \frac{(E_r)_1}{(1 + \frac{2}{5}C_r)}
\]

\[
E_2 = \frac{(E_r)_2}{(1 + \frac{1}{2}C_r)}
\]

\[
F_{DS} = \frac{2}{3}F
\]
where \((E_c)_t\) is the modulus of elasticity of the concrete at time \(t\), \(C_t\) is a creep coefficient at time \(t\), and \(F\) is the force due to differential shrinkage. Assuming that the ultimate creep coefficient of the deck is 2.5 and that the ultimate creep coefficient of the girder is 1.7 for 1-month between casting the girder and casting the slab and 1.4 for greater than 2-months between casting the girder and casting the slab, average ultimate values for the moduli of elasticity are obtained. Assuming 70 percent relative humidity and applying the appropriate correction factors, the recommended ultimate moduli are:

\[
E_1 = 0.5(E_c)_{28d} \quad \text{for the Slab}
\]
\[
E_2 = 0.55(E_c)_{28d} \quad \text{for the girder, age when slab is cast = 1 month}
\]
\[
E_2 = 0.60(E_c)_{28d} \quad \text{for the girder, age when slab is cast ≥ 2 months}
\]

The Recommended Approximate Solution for Practice further simplifies the above equations to allow for a quick and easy solution. This method recommends that the modulus elasticity of the slab, \(E_1\), and the modulus of elasticity of the girder, \(E_2\), be taken as the respective moduli of elasticity for the concrete at 28 days. Then, the final force due to differential shrinkage, \(F_{DS}\), is given by \(F/2\), where \(F\) is the differential shrinkage force calculated with the 28 day moduli of elasticity. The deflection due to differential shrinkage is obtained by multiplying the deflection obtained from typical structural analysis using the 28 day modulus of elasticity by a factor of 4/3.

### 2.7 Types of Continuity Details

The results from recent surveys of designers throughout the United States indicate that the most common continuity detail consists of a cast-in-place continuity diaphragm which provides for the negative moment by allowing the mild reinforcing in the deck to extend over the support and provides for the positive moment by extending either bent prestressing strands or bent mild reinforcing bars from the girders into the diaphragm (Hastak, et al 2003). Various construction sequences are also used, the most common being placing the deck first, waiting several days to a week, and then placing the diaphragm and the remaining small portion of the deck (in the range of 2 to 4 ft on each side of the center of the diaphragm). Virginia has used this type of construction sequence and this type of continuity detail for the majority of its multispans. However, other types of continuity connections and construction sequences have been proposed.
and are used throughout the United States. Many of these different connections have been proposed by Tadros at the University of Nebraska, Lincoln (Ma, et al 1998).

In a paper written at the University of Nebraska, the authors recommend several alternate continuity details based on the results of both analytical modeling and experimental testing of Nebraska University (NU) I-beams (Ma, et al 1998). For the positive moment connection, the authors recommend that extended prestressing strands be used. This recommendation was based on “…the excellent performance of this detail in both the laboratory and in practice…”.

For the negative moment continuity connection, different construction sequences and methods to resist the continuity connection were investigated. First, the conventional system of providing mild reinforcing steel in the deck over the support was investigated. One possible construction sequence is to first place the concrete for the continuity diaphragm only and wait to cast the deck until later. Positive restraint moments will begin to gradually develop over time until the deck is cast. When the deck is cast, rotations at the ends of the beams due to the weight of the slab will counteract the positive moment rotations that have developed due to creep and shrinkage. For this system, it was found that a construction sequence consisting of casting the diaphragm first may lead to cracking of the diaphragm prior to the deck being cast if the deck is not cast within about 230 days from the time that the diaphragm was cast. This limit was based on the assumption that the diaphragm was cast at a girder age of 14 days. Increasing the age at which the diaphragm alone is cast to 28 days will allow the time from casting the diaphragm to casting the deck to increase to about 500 days. Therefore, it was recommended that the diaphragm for this sequence of construction not be placed until the girder was at least 14 days old, and preferably 28 days old, to eliminate the possibility of cracking in the diaphragm due to the positive restraint moments.

A second construction sequence investigated for the conventionally reinforced diaphragm consisted of casting the diaphragm and deck at the same time. For this case, it was found that negative moments due to creep and differential shrinkage would begin to develop in the diaphragm. The more time between casting the girder and casting the deck/diaphragm would produce greater expected negative moments. It is stated that the analysis indicate that these negative moments will be relatively small compared to the elastic moments due to live load and superimposed dead loads. Therefore, it is stated that the time-dependent restraint moments will
not have a significant effect on a system constructed with this sequence of construction. Also, it is stated that this type of system may be “inconvenient” for the contractor. The authors recommend that if this system is to be used, that the beams should be allowed to freely rotate at the ends by providing unbonded joints.

A third construction sequence investigated for the conventionally reinforced diaphragm consisted of casting the diaphragm first, and then casting the deck later. For this case, it was found that significant negative moments may develop over the support. These moments, in addition to the superimposed dead load and live load moments, must be resisted by the mild steel located in the deck. The authors state that “…the only resistance of the continuous beam-diaphragm joint to negative moments is the mechanical interlock due to beam embedment into the diaphragm. This resistance is not quantifiable or reliable.” Therefore, the authors propose that some continuity should be provided in the tops of the beams prior to casting the diaphragm to help carry the tensile forces due to the negative moments. Several methods are discussed which could be used to provide negative moment continuity. In addition to simply providing all of the negative moment continuity reinforcement in the deck, the authors discuss the advantages and disadvantages of using full length post-tensioning. Although this method can provide for better resistance of the negative moments than other methods, it requires the addition of post-tensioning ducts, anchorage blocks, often wider beam widths, and a specialty contractor to tension the strands. A third method discussed to provide resistance to the negative moment is to extend prestressing strands at the top of the girders and couple these strands to the adjacent girder. This method requires that field jacking be done with the use of special couplers. For these reasons, the authors do not recommend using either of these methods for multi-span I-girder bridges.

Instead, the authors present a new method using high strength threaded rods to provide for negative moment continuity. Three specimens were made and tested to determine the effectiveness of resisting the negative moment with either 92.0 ksi or 150.0 ksi rods extending from the tops of the girders and coupled with heavy nuts and rods over the supports. The rods were intended to resist the moments due to the weight of the slab and any construction loads. Additional steel was placed in the deck to resist any additional moments due to superimposed dead and live loads. The authors concluded that this method was effective and recommended its use. They indicated that this method is not only cost effective, but can increase the composite
action of the deck and girder and increase the span capacity by as much as 20 percent.

Prior to presenting the above method for making the negative moment connection, two other methods were developed and proposed at the University of Nebraska (Saleh 1995). The first method used couplers cast into the top ends of the girders in-line with the adjacent girder. A short piece of a high strength threaded bar is then threaded into the coupler and attached end to end with the adjacent bar with the use of a turnbuckle. To accomplish this, one threaded bar would have to have right hand threads while the other would have to have left hand threads. The diaphragm is then cast, providing for some negative moment resistance in the end of the girders. The second method used extended prestressing strands at the ends of the girder to make the negative moment connection. The strands were extended from each girder, field cut, and spliced with the adjacent girder. The strands were then tensioned by jacking against a concrete block which had been cast on the top of each girder. While the strands were tensioned, the diaphragm was placed. Then the jacks were then removed, effectively creating a pretensioned joint.

By performing an economic comparison of the two, it was predicted that the first method would be more economical than the second. Also, since it is expected to be easier to construct, it should be the more attractive of the two methods, even though it does not pre-compress the joint. The second method is expected to be as effective as full length post-tensioning, but should cost less because of the reduced effort in the tensioning operation.

2.8 Other Recommended Practices

In his book Deformation of Concrete Structures, Branson provides an alternative method for determining the restraint moments that will be created due to differential shrinkage and creep of a composite section (Branson 1977). He indicates that the British code allows for the calculation of the “hogging” restraint moments, M, to be calculated at interior supports by using:

\[
M = 0.43DE_1A_1y_{cs}
\]  

(2.65)

where D is the differential shrinkage and creep strain or coefficient, E_1 is the modulus of elasticity of the deck, A_1 is the area of the deck, and y_{cs} is the distance between the centroid of the composite section and the centroid of the deck. The factor 0.43 is said to be a “… reduction coefficient to allow for creep.”

Clarke and Sugie in England gave recommendations for the magnitude for the design
moment of the positive moment connection of precast girders (Mirmiran, et al. 2001). For girders at least 42 in. deep and span lengths varying from 65 to 120 feet, the recommended design value was 550 k-ft. For girders less than 42 in. in depth, the recommended design value was 440 k-ft.

Rabbat and Aswad found that standard details used in Tennessee provided positive moment capacities between 1.19 and 1.70 times the cracking moment (Mirmiran, et al. 2001). They also recommended that a value of 1.20 times the cracking moment be used for the design of the positive moment connection.
3 Research Methods and Materials

3.1 Objectives of the Research

The overall objective of the research is to obtain a better understanding of the behavior of precast, prestressed girders made continuous with a continuity diaphragm. Through a better understanding of the behavior of this type of structural system, recommendations can be made to improve the current design practices. To obtain this overall objective, four specific objectives are considered.

The first objective of the research is to satisfy the requirements of the research proposal in Appendix A. As stated in the proposal, the ultimate goal is to “… develop and test several continuity diaphragm details for PCBT girder bridges, and propose an optimum continuity detail to VDOT.” Six full depth PCBT girders were fabricated and transferred to the Structures and Materials Laboratory at Virginia Tech in order to test three different continuity connections. The research includes tests designed to determine the cracking moment capacity of the different continuity connections. Also, the ability of the connection to remain serviceable when exposed to cyclic loads is investigated.

A second objective of the research is to investigate the influences of the creep, shrinkage, and temperature on the behavior of the continuous structural system. Shrinkage specimens for the deck concrete were made and monitored for each test. Thermocouples were placed within the girders and the deck to monitor changes in temperature during the curing process. The end reactions were closely monitored during the deck curing phase so that the influences of the heat of hydration and shrinkage could be better understood. In the most recent work performed for the NCHRP 519 project, it was found that the curing process has a significant influence on the development of restraint moments which has generally not been considered (Miller, et al 2004).

A third objective is to investigate the effective continuity that can be achieved with the continuity connection. End reactions and member deflections were recorded for every test to determine the ability of the connection to transfer moments. The ability of the connections to transfer loads over an expected 10,000 thermal cycles was also investigated.

A fourth objective is to determine the structural model that best predicts the behavior of the composite structural system.
3.2 Analytical Review

An analytic review was performed in order to predict the restraint moments that may occur in typical girder and slab systems using the PCBT sections provided by VDOT in order to determine what girder and slab system should be tested. Five different methods were used to predict these moments. Details of the prediction models and a design example are in the paper in Appendix C. The predicted moments were used to establish the design moments used for the experimental study.

Originally, the PCBT-37 section was investigated for different span lengths and girder spacing. Once it was determined that the fabricator would be unable to provide this size section, PCBT-45 sections were investigated. It was found that the smallest beam spacing of 6 ft-0 in. would create the largest positive moments. This makes sense since it is expected that the larger the slab, the greater will be the differential shrinkage and creep due to dead loads. Since both of these loads reduce the positive moment at the continuity connection, the smallest beam spacing will cause the largest positive moment.

It was decided to use a more typical number of strands instead of using the absolute maximum number of strands that could be placed in the section. A PCBT-45 section with 28 strands was found to work well at a span length of 80 ft. VDOT provided preliminary design tables which indicated that for a 28 strand pattern, a span length of 85 ft could be achieved. The information on the preliminary design sheet was checked and found to be correct; however, it was decided to model the slightly shorter span arrangements because the shorter spans will provide for the potential for more positive moment (for a given number of strands).

Therefore, the analytic review resulted in a model of a two span continuous system, each span being 80 ft in length, with a PCBT-45 precast, prestressed girder and a 7 ½ in. slab that is 6 ft-0 in. wide. A 1 ½ in. bolster was assumed to be present over the top of the top flange.

Based on current recommendations, the positive and negative cracking moments were calculated using gross properties of the composite section. The composite section used for the calculations was based on the full width of the deck (6 ft-0 in.) with a modular ratio of 1.0. The full area of the girder with a 1 ½ in. bolster was also used. The cracking stress of the concrete for determining both the positive and the negative cracking moments was taken as 7.5 times the square root of the 28 day compressive strength of the diaphragm concrete, 4000 psi.
Moments due to creep, shrinkage, and temperature were calculated and averaged for the different methods. The live loads were determined using SAP2000 assuming a two span continuous structure with 80 ft-0 in. spans. Live loads are shown for the negative case only, but were checked for different span arrangements for the positive case and found not to control. Both HS-20 and Lane Loading were investigated to produce the maximum loading. The composite dead loads provided by VDOT were reduced by a factor of 2 so that they would not reduce the predicted positive moments. A summary of the design loads that were used for testing follows in Table 3.1.

Table 3.1 – Design Moments for Specimens

<table>
<thead>
<tr>
<th>Type</th>
<th>Calculated Value</th>
<th>1.2</th>
<th>1.0</th>
<th>0.75</th>
<th>0.5</th>
<th>0.25</th>
</tr>
</thead>
<tbody>
<tr>
<td>Positive</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cracking</td>
<td>523.8</td>
<td>628.6</td>
<td>523.8</td>
<td>392.9</td>
<td>261.9</td>
<td>131</td>
</tr>
<tr>
<td>Ult. Cap. – Test 1</td>
<td>711.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ult. Cap. – Test 2</td>
<td>773.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Restraint</td>
<td>205.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Thermal</td>
<td>441.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cracking</td>
<td>-919.1</td>
<td>-1102.9</td>
<td>-919.1</td>
<td>-689.3</td>
<td>-459.6</td>
<td>-229.8</td>
</tr>
<tr>
<td>Ult. Cap. – Test 1</td>
<td>-2653.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ult. Cap. – Test 2</td>
<td>-2653.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Restraint</td>
<td>-1126.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Thermal</td>
<td>-104.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Live Load</td>
<td>-531.3/0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.3 Overview of Materials and Equipment

The Fabrication Phase began when six full depth, Precast Concrete Bulb Tee (PCBT) specimens, each 15 ft 4 in. long and 45 in. deep, were fabricated with different positive moment end connections in order to test the strength and serviceability of three different continuity connections. The specimens were named VT1 through VT6. At the fabricator, vibrating wire gages (VWG) and thermocouples were installed and monitored with a Campbell CR23X data collection system. The gages were monitored throughout both the steam curing process and detensioning of the strands.

The six specimens were then transported to the Structures and Materials Laboratory at Virginia Tech, where four were placed in temporary storage while girders VT1 and VT2 were assembled for Test 1. The two specimens each had their bottom layer of 12 – ½ in. diameter
grade 270 prestressing strands extended and bent upwards. Once placed end to end, formwork and reinforcing steel were placed to connect the specimens with a diaphragm and a continuous 6 ft wide and 7 ½ in. deep deck was cast, beginning the Static Phase. Electrical resistance gages in the deck, vibrating wire gages in the girders, and end reactions were monitored in order to document the effects of the heat of hydration and shrinkage of the deck on the composite system. After a week of curing, the formwork was removed and the system was monitored until the deck concrete reached at least its design strength of 4 ksi.

The Service and Cyclic Loading Phase began when transfer beams were placed across the top of the specimens at the passive end and the center and bolted down to the reaction floor. The support for the active end was removed and a frame with an MTS actuator was moved in place over the support. The actuator was attached to the end of the specimen with collar beams and a series of service and cyclic tests were performed to determine the cracking moment capacity of the composite system at the diaphragm and the ability of the system to maintain integrity over 10,000 load cycles. Finally, the system was taken to near failure.

Test 2 was similar to Test 1 except that instead of extended prestressing strands extending from the bottom of the specimens, extended 180° bent No. 6 reinforcing bars were extended from the ends of the girders. A continuity diaphragm and a 6 ft wide deck were constructed, and the system was monitored and tested similar to Test 1. Test 3 was different because no continuity diaphragm was constructed. The girder specimens were placed end to end approximately 3 in. apart, and the 6 ft wide deck was cast continuously over both. The system was still monitored for changes in end reactions, and the testing up to 10,000 cycles was performed, mainly to document the onset and the increase in damage in the deck due to cracking.

More details on the materials and equipment used are provided in the following sections. To serve as a quick reference, Table 3.2 shows a listing of the equipment used. Figure 3.1 shows the timeline of events for all phases of testing for all three tests. Figure 3.11 shows details of the test setups.
<table>
<thead>
<tr>
<th>Material/Equipment</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Barrells</td>
<td>TransTek Model 354-0000 DC-DC Gaging Transducer with a working range of +/- 1.00 in. Linearity +/- 0.5 % Full Scale over total working range.</td>
</tr>
<tr>
<td>Bearing Pads</td>
<td>50 Durometer neoprene, 1.9375x10x24 Laminated Pad with 2-11 gage shims, manufactured by Scougal Rubber. (206) 763-2650.</td>
</tr>
<tr>
<td>Campbell CR23X</td>
<td>Campbell Scientific Model CR23X Micrologger with 2 Multiplexers for reading the Vibrating Wire Gages.</td>
</tr>
<tr>
<td>Deck Concrete</td>
<td>A-4 General Use Concrete Provided by ConRock. Headquarters at Construction Materials Company, P.O. Box 1347, Harrisonburg, VA, 22803-1347, Phone (877) 266-7625.</td>
</tr>
<tr>
<td>Extensometer, 2 in.</td>
<td>Model SE2-50 extensometer calibrated on 6/16/04 attached to Satec UTM. Used for testing bars and strands.</td>
</tr>
<tr>
<td>Girders</td>
<td>Six test girders manufactured in the Chesapeake, VA plant of Bayshore Concrete Products Corporation, P.O. Box 230, Cape Charles, VA 23310.</td>
</tr>
<tr>
<td>Mitutoyo Dual Directional Digital Counter</td>
<td>Adjustable digital counter with an accuracy of 0.001 in. and a range of 0 to 12 in. used to manually calibrate instruments.</td>
</tr>
<tr>
<td>MTS Ram</td>
<td>MTS Single Ended Actuator, Model 243.60, Force of 235 Kips Compression, 150 Kips Tension. Static and Dynamic Stroke of 20 in.</td>
</tr>
<tr>
<td>MTS System</td>
<td>MTS 458.10 MicroConsole used to operate the MTS ram.</td>
</tr>
<tr>
<td>Reinforcing Steel for Deck</td>
<td>Provided by RESCO Steel Products Corporation, Subsidiary of Roanoke Electric Steel Corp., 438 Kessler Mill Road, Salem, VA, 24153. Phone (540) 387-0284.</td>
</tr>
<tr>
<td>Satec System UTM</td>
<td>Model M300WHLV Satec System Universal Testing Machine controlled with the MATS II Controller. Nuvision II software V 04.01.001 used for controlling the machine.</td>
</tr>
<tr>
<td>Self-Leveling Compound</td>
<td>Rockite fast-setting hydraulic type cement.</td>
</tr>
<tr>
<td>Strain Gage Indicator</td>
<td>Micro-Measurements P-3500 Strain Indicator.</td>
</tr>
<tr>
<td>StrainSmart</td>
<td>Data collection software, StrainSmart Data Systems, Version 3.10, Build 3.1.14.490</td>
</tr>
<tr>
<td>Switch and Balance Unit</td>
<td>Micro-Measurements SB-10 Switch and Balance unit.</td>
</tr>
<tr>
<td>System 5000</td>
<td>Model 5100B Scanner by Vishay Measurements Group, Inc.</td>
</tr>
<tr>
<td>Thermocouple Reader</td>
<td>Omega Model HH21 Microprocessor Thermometer.</td>
</tr>
<tr>
<td>Thermocouple Switch Box</td>
<td>Homemade box using an Omega 2-Pole switch, Omega Part SW142-24-B-PG.</td>
</tr>
<tr>
<td>Thermocouples</td>
<td>Omega Thermocouple wire type TT-T-20-SLE, +Blue Copper, -Red Constantan with type NMP mini connectors.</td>
</tr>
<tr>
<td>Vibrating Wire Gage</td>
<td>Geokon Model 4200. Temperature range of -20 °C to +80 °C. Active gage length of 153 mm with a sensitivity of 1 με. Thermal Coefficient of Expansion of 12.0 ppm/°C.</td>
</tr>
<tr>
<td>Vibrating Wire Gage Reader</td>
<td>Geokon Model GK-403 Vibrating Wire Readout set to position &quot;D&quot;.</td>
</tr>
<tr>
<td>Vibrating Wire Gage Switch Box</td>
<td>Homemade box using an Omega 4-Pole switch, Omega Part SW144-12-B-PG.</td>
</tr>
<tr>
<td>Whittemore Gage</td>
<td>Humboldt Model H 323OD Gage with Mitutoyo electronic digital dial gage. Reads in inches to the nearest 0.0001 in.</td>
</tr>
<tr>
<td>Wirepots</td>
<td>Celesco Position Transducer Model No. PT101-0010-111-5110 with a 10 in. measurement range. Position Signal Sensitivity 0.20371 mV/V/in.</td>
</tr>
</tbody>
</table>
Experimental Procedures

3.4.1 Fabrication Phase

The six girder specimens, named VT1 through VT6, were fabricated by Bayshore Concrete Products, Inc., at their Chesapeake, Virginia plant. The shop drawing showing the complete bed layout and girder details is included in Appendix D. A typical sequence of construction usually begins with jacking the strands and placing the reinforcing steel within the bed on the first day. Usually within 24 to 48 hours from jacking the strands, the concrete for all members within the bed is placed. Steam curing begins and occurs for 12 to 18 hours, the forms are stripped and the strands are released the following day. Often, the entire process can occur within a time period of two days. For this project, several things occurred during the fabrication phase that were not typical for the construction of prestressed girders, see the Timeline of Events in Figure 3.1. First, the six specimens were only part of the pour; three larger beams were included in the same bed. The concrete for the beam specimens was cast one day and the
concrete for the other beams was cast the following day. The strands were approximately four
days old at the time of detensioning (the concrete was 2.1 days old), instead of the typical two
days old. Type II cement was used because of the scarcity of the Type III cement which is
typically used. Table 3.3 shows the mix design for a 5.0 cubic yard mix used for construction of
the girders. Newcem is a slag cement, Daravair is an air entraining agent, Sika 6100 is a
superplasticizer, Plastiment is a water reducing admixture, and DCI-S is a corrosion inhibitor.

<table>
<thead>
<tr>
<th>Material</th>
<th>Quantity</th>
<th>Admixture</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>6187 lbs</td>
<td>Daravair 30 oz</td>
<td></td>
</tr>
<tr>
<td>#57 Stone</td>
<td>9040 lbs</td>
<td>Sika 6100 233 oz</td>
<td></td>
</tr>
<tr>
<td>Type II Cement</td>
<td>2325 lbs</td>
<td>Plastiment 117 oz</td>
<td></td>
</tr>
<tr>
<td>Newcem</td>
<td>1550 lbs</td>
<td>DCI-S 1280 oz</td>
<td></td>
</tr>
<tr>
<td>Water</td>
<td>94 gals</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3.3 – Mix Design for Girder Concrete

To better document the events that took place during the Fabrication Phase, a log is
provided detailing the most important aspects of this phase.

**Monday, June 21, 2004,** per Mr. Armstrong, a QC employee at Bayshore, the strands
were pulled and jacked to 32 kips in the evening.

**Tuesday, June 22, 2004,** all instrumentation was placed in the 6 specimens starting at
about 9:30 a.m. The lead wires for the vibrating wire gages and thermocouples were run to the
ends of each of the beams. Once the forms were placed, the wires were run underneath the rail
for the external vibrator and taken to a central location between beams VT3 and VT4.
Thermocouples left to monitor the bed temperature were placed in VT1 and VT3. The one in
VT1 was placed on top of the beam and the one in VT3 was placed on the side of the beam.

Since rain was predicted for the evening, and the clips had not yet come for the other
three beams in the pour, the pour was delayed. Also, the wires were not connected to the
Campbell Data Collection system since rain was imminent. It did rain later in the evening.

Figure 3.2 shows the end of girder VT6 and the bulkhead used to hold the prestressing
strands. Figure 3.3 shows a Vibrating Wire Gage (the blue gage) installed 8 in. from the end of
the girder and 4 in. down from the top of the girder. Per the manufacture’s recommendation, the
gage is hung with plastic zip ties and held into place with wood spacer blocks.
Wednesday, June 23, 2004, the Campbell Data Collection system was wired from about 7:00 a.m. to about 8:00 a.m. It was turned on and left to run from this point. Figure 3.4 shows the Campbell CR23X fully wired and ready to record the data. The unit was set to record at ten minute intervals. The unit used two multiplexers to record 31 vibrating wire gages. Each of the 6 girders contained 5 gages, and one gage was located in the control cylinder. Nine thermocouples, 4 at girder VT1 and 5 at girder VT3 were also recorded.

The first batch of concrete arrived for placement in the far end of beam VT6 at 12:51 p.m. The concrete had an 8 in. slump, 5 ½ percent air content, and was 89 degrees Fahrenheit at the time of placement. Beams VT6, VT5, and VT4 were placed from about 12:51 p.m. to about 1:15 p.m. Air temperature was measured to be 88 degrees Fahrenheit. It was partly cloudy with variable winds. Approximately 2 or 3 Tuckerbuilts were used for this first pour.

Beams VT3, VT2, and VT1 were placed from about 1:15 p.m. until about 2:15 p.m. Sampling near the end of this pour indicated the concrete had a 3 in. slump, 3.8 percent air content, and was 90 degrees Fahrenheit at time of placement. Air temperature was measured to
be 90 degrees Fahrenheit. It was mostly cloudy with gusty winds of about 10 to 15 miles per hour. Near the end of placing concrete in VT2 and VT1, the concrete in the Tuckerbuilt prematurely set and had to be discarded. It was indicated that this was a short batch instead of a full batch. The concrete in the beams became fairly stiff during the delay which followed.

Concern for the potential of a cold joint developing was expressed during casting, but one was never noticed. The 6 beams were covered with insulated tarps soon after the raked surface was made in the top surface.

Test cylinders and beams were made during the placing operation. These specimens were numbered in the order in which they were made, and an attempt was made to make representative cylinders from each batch. The concrete for the specimens was taken directly from the Tuckerbuilts into a wheelbarrow and the specimens were made as quickly as possible.

A discussion with Larry McCallum of Bayshore Concrete Products and a representative of Sika chemicals took place. It was indicated that the mix being used was not the same mix that had recently been used at that plant for PCBT girders made for the Route 17 project in Chesapeake. A different superplasticizer manufactured by Sika was being used. Also, the cement type was different than normally used. Per the plant manager, Ralph Bruno, it had become too difficult to get the type III cement usually used. Therefore, type II cement was being used. Mr. McCallum indicated that this type II cement used had similar setting properties of type III cement.

A preset was used for determining when the steam would be turned on. An employee indicated that the steam was turned on at 6:45 p.m. The steam pipe entered between beams VT3
and VT4. Therefore, these beams saw the most steam.

**Thursday, June 24, 2004,** at 6:30 a.m., Bayshore broke two cylinders. The specimens called VT1 came from the first batch of concrete made which was placed starting at beam VT6 and the specimens called VT2 came from the last batch of concrete made which was placed in beams VT1 and VT2. Compressive strengths of 3182 psi and 4375 psi from VT1 and VT2 respectively were obtained. These breaks occurred at about 12 hours of steam curing. At about 8:45 a.m., Bayshore broke two more cylinders. Specimens VT1 and VT2 broke at 3798 psi and 5462 psi respectively. These breaks occurred at about 14 hours of steam curing. Two more breaks were done in the afternoon. VT1 and VT2 broke at 6842 and 8592 respectively. The exact time these breaks were made could not be determined.

The formwork for the remaining three beams in the pour had to be partially removed since the rain the night before had ruined the cardboard inserts. The process of removing and reinstalling the forms took place from about 9:00 a.m. to 11:00 a.m. At 11:33 a.m., concrete began to be placed in the far beam with external vibration occurring at the same time. Placement occurred in the other beams until about 1:30 p.m.

**Friday, June 25, 2004,** the steam was ordered to be ramped down starting at 8:30 a.m. At about 10:30, the steam was ramped down more and shortly after left at just a trickle. At 11:00 a.m. the steam was ordered to be completely shut off. At about 12:00 p.m., the tarps were removed from beams VT1-VT6. All cylinder and beam specimens were removed at this time and placed in the truck for transporting back to Blacksburg. Removal of the formwork began at the far end at about 1:00 p.m. During this time, the wires connected to all beams except VT3 were removed from the Campbell unit and pulled back through from behind the vibration rail. This was done so that the formwork could be removed. Once the wires were removed, they were run along the top flange of the beams and reconnected to the Campbell unit.

At about 3:00 p.m., with thunderstorm warnings in effect and a tornado warning in the adjacent city, detensioning of the strands began. The top 4 partially prestressed strands (taken to 1 or 2 kips) were first cut loose at the abutments, and then the 4 draped strands (draped in the other three beams, but straight in beams VT1 through VT6) were cut loose at the abutments. Next, the top 8 strands (4 draped and 4 partially prestressed) were cut loose between the beams (8 locations). This operation was completed at about 3:30 p.m. Detensioning of the bottom
strands (24 total) began at about 3:35 p.m. at the abutments. The top layer of 12 was cut at the abutments with alternate left to right strands cut from the outside in. After the top layer, the bottom layer was cut in a similar fashion. Heavy lightning halted the operation at this point.

Most of the workers left for the day, but one stayed around to finish detensioning the strands. Starting at VT6, which was at the far abutment, the bottom 24 strands were cut completely releasing beam VT6 at 4:14 p.m. VT5 was released at 4:19 p.m. VT4 was released at about 4:24 p.m. VT3 was released at about 4:32 p.m. Next, the near end of VT1 was released. Finally, the connection between VT1 and VT2 was released and completed at about 4:44 p.m. Three samples of the prestressing strand were obtained. Shortly after the final release was made, a 4x8 cylinder was tested to determine the modulus of elasticity at time of release. Using the testing machine at the yard, two tests were run and the modulus of elasticity was calculated to be 5,193 ksi and 5,250 ksi. A camber of 1/16 in. was measured for each of the beams.

The instrumentation was allowed to run for about another hour, and then the Campbell unit was disconnected. The wires coming out of the beams were coiled and zip tied to the extended bars. All tags were marked again to ensure that the markings would remain legible. Figure 3.5 above shows a completed girder specimen prior to detensioning.

3.4.2 Static Phase

The purpose of the Static Phase testing was to monitor and record early age changes in the reactions of the two girders made continuous in order to estimate the magnitude of the early age restraint moments that develop. The electrical resistance gages in the deck as well as the
vibrating wire gages in the girder were recorded to determine the changes in strain. Thermocouples placed within the girders and the deck were monitored so that the magnitude of the thermal gradients could be determined. Two shrinkage specimens were cast with the deck concrete in order to monitor shrinkage of the deck. To confirm movements, dial gages were placed under the girders and monitored.

Figure 3.6 shows the test setups used for the Static Phase. A main concern that existed prior to starting the testing in this phase was the overall side to side stability of the beams during casting the deck and testing. A single load cell under each reaction would not provide enough stability. Therefore, it was decided to construct the Four Column Stools, see Figure 3.18. Eventually it was determined that the Four Column Stools were not working accurately enough to use, so it was decided to use the stools under the center reactions and a combination of the stools and two load cells at the end reactions. Under the passive end, two 50 kip load cells were placed and under the active end, two 10 kip load cells were placed. With this arrangement, the overall stability seemed adequate and the end reactions could be measured accurately. A second concern prior to starting this phase was the eventual removal of the test specimens after testing. The two overhead cranes in the Structures and Materials lab each had a capacity of 5 tons (giving a total of 10 tons capacity) and the total test setup once the girders were connected was estimated to weigh approximately 22 tons. It was anticipated that the each test would be jack hammered apart after completion and removed in sections. However, it was ultimately decided to try to reduce the weight enough so that the two cranes could pick up half of the test specimen and remove the total test in one piece. To reduce the weight, the slabs were not cast as far out to the exterior ends of the girders as originally planned. Instead, the slabs were cast approximately 9 ft 6 in. from the interior ends of the girders. Test 1 and Test 2 were removed in one piece while Test 3, since it did not have a continuity diaphragm connecting the two girders, was jack hammered apart after testing and the two girder/deck sections were removed individually.

Once the beams were set in place, formwork was installed as shown in Figure 3.7. To follow the same procedures used by most contractors, the deck forms were supported from the girders and the continuity diaphragm forms were supported from the reaction floor. A 3/8 in. diameter hole was drilled at 2 ft on center into the edge of the top flange and a 3/8 in. by 3 ¾ in. long Powers Power Stud anchor was driven into the hole. The anchor was then bolted through the 2x6 to vertically support the formwork. For side to side support, ¼ in. diameter Pencil Rods
Figure 3.6 – Static Phase Test Setups
at 2 ft on center with clamps were used. This material worked well because it was inexpensive and could be cut to any length needed. Once the formwork was in place, the reinforcing steel for the deck was placed. The steel was supported off of high chairs and tied together with wire ties.

End forms were constructed of plyform with holes drilled at the location of the longitudinal steel to so that the steel could extend beyond the ends of the slab. The top two center bars were gaged with electrical resistance wire gages. A maximum number of gages that could be read by the two switch and balance units were installed. Near the center, two gages were placed side by side in order to obtain a backup reading in case one of the gages failed. Three thermocouples were placed in the center of the active span above the center of the girders and three were placed in center of the active span 3 in. from the outside edge. The thermocouples were placed in the center of the deck vertically and ½ in. from the top and bottom of the deck.

In preparation for casting the deck, the vibrating wire gages were attached to the homemade vibrating wire gage switch box and the electrical resistance gages were attached to the two switch and balance units. The load cells and the four column stools were also attached to the switch and balance units.

Table 3.4 shows the mix design for the deck concrete. The deck concrete for all three tests was the same mix, a Class A-4 General Use mix with a target slump of 2-4 in. and a target air content of 6 +/- 1 percent. Sika AE is an air entraining admixture. To improve workability, the actual slump used for all tests was 5 +/- 0.5 in.
Table 3.4 – Mix Design for Deck Concrete

<table>
<thead>
<tr>
<th>Material</th>
<th>Quantity</th>
<th>Material</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>1317 lbs</td>
<td>Pozzolans</td>
<td>317 lbs</td>
</tr>
<tr>
<td>#57 Stone</td>
<td>1741 lbs</td>
<td>Water</td>
<td>34.3 gals</td>
</tr>
<tr>
<td>Type I/II Cement</td>
<td>318 lbs</td>
<td>Sika AE</td>
<td>2-4 oz</td>
</tr>
</tbody>
</table>

Figure 3.8 shows an overall test setup with the formwork in place for Test 2. The setup for Test 1 and Test 2 were similar, with the only difference being the differences in the continuity connection. For Test 1, the bottom level of 12 prestressing strands were extended 9 in. out and bent up at a 90 degree angle an additional 15 in. The beams were placed 12 in. apart and the continuity diaphragm was cast even with the ends of the girders. For Test 2, 4-No. 6 mild reinforcing bars each bent 180 degrees extended from the end of the girders 11 in. The beams were placed 13 in. apart and the continuity diaphragm was cast even with the ends of the girders. For Test 3, no continuity diaphragm was provided at the interior supports. The girders were simply placed approximately 3 in. apart and the deck was cast continuous over the opening.

Figure 3.9 shows a close up of the continuity diaphragm with part of the formwork in place and the different connections used for (from left to right) Test 1 and Test 2.

Figure 3.10 shows details of the reinforcing which was used for both the deck and the continuity diaphragm. The notation used is typical VDOT notation, where the first two letters indicate the type of bar (DL is deck, longitudinal), the second two numbers indicated the size of bar (04 is a No. 4 bar), and the last two numbers indicate the individual bar mark.
Figure 3.9 – Details of Continuity Diaphragm

- 2-DSO401: min. 16" Lap
- 5'-3" 3'-9" 1 1/2"
- 2-DL0601 - 4 Eq. Spa.
- BS0402
- BS0401
- BS0403
- BLO501 (4 Typ.)
- BLO501 (2 Typ.)
- 24 Straight Strands
- 18 - #5 Long. Bars
- #5 @ 7 1/2" O/C

Typical Test Section

- Deck Reinforcing Not Shown
- 2 DS04
- DL0601

Side View - Test #2

Diaph. Reinf. Test #1

- 2-DSO401: min. 16" Lap
- 1'-1"
- 4 Straight Strands
- 3" 2 24 Straight Strands
- BS0403
- BS0402
- BLO501 (4 Typ.)
- BLO501 (2 Typ.)

Diaph. Reinf. Test #2

- 2-DSO402: min. 16" Lap
- 5'-2"
- 4 1/2" 11" 11"

Figure 3.10 – Reinforcing Details for Tests
3.4.3 Service and Cyclic Loading Phase

The purpose of the Service Loading Phase for Test 1 and Test 2 was to determine the positive moment capacity at the girder and diaphragm interface that would cause initial cracking, $M_{cr}$. Since positive moment connections are more difficult to construct than negative moment connections, the first cracking at the interface due to positive moment was investigated. Also, the ability of the diaphragm connection to resist service moments of at least 1.2 times the anticipated cracking moment of the section was investigated. For Test 3, the deck was investigated to determine the magnitude of the negative moment that would cause first cracking. The purpose of the Cyclic Loading Phase was to subject the connections to 10,000 cyclic loads caused by positive and negative thermal gradients and to measure the degradation of the connections over the cycles. It was anticipated that most transportation structures would not encounter more than 10,000 maximum thermal cycles throughout a 75 year service life.

Figure 3.11 shows the typical setup for each of the three tests. After the deck and diaphragm concrete had reached at least 4,000 psi, the end blocks were placed on both the active and passive ends and 9x14x2 in. elastomeric bearing pads were placed above the supports at the interior of the setup. At the passive end, a W18x76 transfer beam was placed across the top of the end block and bolted down to the reaction floor beam with 4-1 in. diameter all threaded rods at each end. In the center of the setup, spacer plates were placed on top of the elastomeric pads and a W10x33 short transfer member was placed longitudinally on top of the spacer plates. A second W18x76 transfer beam was then placed perpendicularly across the top of the W10 transfer member and bolted down to the reaction floor beam also with a total of 8-1 in. diameter all threaded rods. At the active end, two hydraulic jacks were placed under the girder and the end was lifted just enough to remove the active end bearing assembly. The frame supporting the MTS ram was then moved into position so that the ram was directly over the bearing at the active end. Two W10x88 collar beams were then clamped to the end of the beam with 2-1 in. diameter all threaded rods on each side. The top collar beam was bolted to the bottom of the MTS ram, thereby attaching the active end of the beam to the ram. The active end was temporarily supported with the jacks until testing began.

Figure 3.12 shows an overall setup for Test 3. Prior to testing, the wires for the electrical resistance gages were removed from the switch and balance units and attached to the System 5000 data recorder. Also, the wirepots at the ends of the beams and at the center of the
diaphragm were installed and attached to the System 5000. The barrels, also known as LVDTs, were installed at approximately 1 in. from the bottom of the end of beam and diaphragm interface for Test 1 and Test 2. For Test 3, the barrels were installed at the bottom of the interior ends of the beams and near the top of the web for the beams. The barrels and the cable attached to the MTS system were also attached to the System 5000. Figure 3.23 shows the LVDTs in place at the bottom of the girder/diaphragm interface for Test 3.
Since the MTS foot, the W10x88 collar beams, and the all threaded rods were hung below the load cell on the MTS system, a considerable amount of weight, approximately 1.91 kips, had to be zeroed out prior to testing. A correction for the amount that was zeroed out was made to the target test values prior to each test. This way, the load applied by the MTS system would be the actual load desired. This correction has already been made in all of the tests results. The following steps outline the typical procedure that was used for each of the Service and Cyclic tests.

1. The thermocouples were read with the HH21 microprocessor and the shrinkage specimens were read with the Humboldt Whittemore gage.
2. For Test 1 and Test 2, the valve controlling the hydraulic jacks was released causing the active end to cantilever.
3. The MTS system was turned on using DC load control and taken to a load of 0.0 kips.
4. The dial gages, the load cells attached to the P-3500, and the vibrating wire gages were read and recorded while the MTS system held the end at a 0.0 kip load reading.
5. The System 5000 data recorder and the StrainSmart software were turned on. All sensors except the MTS deflection were zeroed. The strain gages were then shunt-calibrated and a new scan session was started. The initial zero reading point was then recorded.
6. For the Service Phase tests, the load on the MTS system was increased to the first recording point and all sensors were recorded with the software. Also, the dial gages, load cells, and vibrating wire gages were recorded.
7. For the Cyclic Phase tests, the MTS system was set to run the first 1000 or 2000 cycles. The DC set point was set to the average of the predetermined load range, the 458.90 function generator was set to Sine Wave, and the frequency adjust knob was set to produce a frequency of 0.25 Hertz. The program was then set to Run.
8. After the cycles were complete, the set point was taken back to 0.0 kips and all the sensors were recorded. The active end of the specimen was then loaded to produce certain percentages of the cracking moment, such as 0.25, 0.5, 0.75, 1.0, 0.5, 0.0 and the sensors were recorded at the different load values.

9. After the final reading, again with a DC set point load of 0.0 kips, the hydraulic jacks were pumped up to support the active end and the MTS system was turned off.

Since there was no continuity connection for Test 3, the active end could not be allowed to cantilever. The jacks were left in place until the MTS system was turned on and allowed to support the active end. For the first two tests, load control was used and the end was loaded up to a given load. For the remainder of the tests, displacement control was used, allowing the active end to be moved up and down a predetermined displacement.

3.5 Material Testing

3.5.1 Concrete Testing

The concrete material testing had two main purposes. First, tests were performed to determine the actual compressive strength, $f'_c$, the split tensile strength, $f_t$, the flexural beam strength, $f_r$, and the modulus of elasticity, $E_c$, at different ages for the girder and the deck concretes so that the actual values of these properties could be used in calculations. Since first cracking of the concrete was an important aspect of this study, a second purpose was to investigate the relationship between actual compressive strength and the tensile strength of the concrete.

All of the specimens made at the fabricator were placed alongside the girders so that they would receive the same steam curing that the girders received. The specimens made for the decks were covered with plastic and kept covered for 7 days, the same amount of time that a conventional deck is covered. At the time that the prestressing strands were released, the cylinders made for the girders were transported from the fabricator to Virginia Tech where they were stored outdoors.

Test cylinders and beam specimens were made in accordance with ASTM C 31 (2003). The 4 in. by 8 in. cylinders were made in two equal lifts, rodding each lift 25 times with a 3/8 in.
diameter rod. The 6 in. by 12 in. cylinders were made in three equal lifts, rodding each lift 25 times with a 5/8 in. diameter rod. The 6 in. by 6 in. by 20 in. flexural beam specimens were made in 2 lifts, rodding each lift 60 times with a 5/8 in. diameter rod. The 6 in. by 6 in. by 11 in. shrinkage specimens were made in 2 lifts, rodding each lift 33 times with a 5/8 in. diameter rod. In addition to rodding, the specimens made for the girders were tapped with a rubber mallet 10 to 15 times per layer.

Compressive strength tests were done in accordance with ASTM C 39 (1986). All of the specimens made for the girders up to the 28 day breaks were sulfur capped in accordance with ASTM C 617 (2003). Specimens were tested in the Forney 400 kip hydraulic compression machine. Figure 3.13 shows the Forney machine with a flexural beam specimen. The rate of loading was adjusted to approximately 35 psi/sec. This rate will cause a 4 in. diameter cylinder at 4 ksi to fail in just less than 2 minutes, and a 4 in. diameter cylinder at 8 kips to fail in just less than 4 minutes.

The splitting tensile strength tests were done in accordance with ASTM C 496 (2003). Figure 3.14 shows specimens prepared for the typical tests. From left to right, two cylinders are sulfur capped and ready for compressive strength testing. Next, a specimen has the collar attached which is used for determining the modulus of elasticity. In the back on the right is a flexural beam specimen. In the front on the right is a specimen in the splitting tensile strength apparatus. Notice the two thin strips of wood used to cushion the specimen from the steel apparatus used for the tensile strength testing. The specimens were loaded at a slower rate than the compressive strength specimens. Once failure occurred, the maximum load P was recorded. The splitting tensile strength, $f_t$, is then determined as a function of the load P, the length of the cylinder, L, and the diameter of the cylinder, d as in the following:
The tests for determining the modulus of elasticity were done in accordance with ASTM C 469 (2003). First, a compressive strength test was performed, and forty percent of the compressive strength was used to set the maximum load to perform the modulus of elasticity test. This is done to ensure that the concrete is tested in the linear elastic range. To perform a test, the collar was placed on a specimen and the specimen was then loaded to the maximum load (forty percent of the compressive strength) and unloaded to set the collar. The initial dial gage reading was then recorded and a value of 0.0005 in. was added to give the first reading point. The load at the first reading point was recorded and the specimen was loaded up to the maximum load where a second reading of the dial gage was taken. Since the collar pivots about the center, the actual strain in the specimen is ½ the recorded strain. By knowing the initial and final loads, $P_i$ and $P_f$, the area of the cylinder, $A$, the initial and final dial gage readings, $D_i$ and $D_f$, and the gage length of the collar, $G_L$, the modulus of elasticity, $E_c$, can be determined from:

$$
E_c = \frac{(P_f - P_i)}{\left(\frac{A}{(D_f - D_i)}\right) G_L^2}
$$

(3.2)

The tests for the flexural beam specimens were done in accordance with ASTM C 78 (1984). Figure 3.15 shows a flexural beam prepared for testing. Beam specimens of the concrete used in the girders were made 6 in. by 6 in. by 20 in. For testing, the formed surface was turned sideways and actual dimensions were taken. The specimen was then loaded in the Forney with a third-point loading to produce a region of constant moment in the center. The modulus of rupture, $R$, was then determined by the maximum load, $P$, the length of the specimen,
L, the average width, b, and the average depth, d as:

\[ R = \frac{PL}{bd^2} \]  

(3.3)

### 3.5.2 Shrinkage Testing

Shrinkage specimens were made for each of the three decks. To better match the volume to surface ratio of an actual deck, the specimens were made 6 in. by 6 in. by 11 in. After removal from the forms, the sides and ends of the specimens were sprayed with a sealant to prevent evaporation. Whittemore points or Demec points were installed at an 8 in. gage length to monitor the shrinkage of the specimens during the static phase.

For Test 1, Whittemore Points were installed in the shrinkage specimens while the concrete was still plastic. The concrete was then allowed to cure the full 7 days prior to taking the first readings. It was decided that readings prior to 7 days would be useful, therefore, for Test # 2, the Whittemore Points were installed while the concrete was still plastic, but readings commenced as soon as the concrete had gained sufficient strength enough to ensure that the points were secure in the concrete. Since difficulty was encountered taking the readings for both of the first two tests, Demec points were attached to the surface of the specimens for Test 3 with epoxy. These points have a much smaller hole in the center which was found to enable a more consistent reading with the Whittemore gage. Readings commenced for Test # 3 as soon as the concrete and the epoxy had set.

### 3.5.3 Mild Steel Testing

The purpose of the mild steel testing was to determine the actual properties for a sample of the reinforcing bars used in the deck. Testing was done in accordance to the requirements of ASTM A 370 (1997). The yield strength, \( F_y \), is considered as the first stress at which an increase in strain occurs without an increase in stress. The tensile strength, \( F_u \), is considered as the
maximum load obtained during testing divided by the original cross sectional area. The elongation is considered as the percent increase in length of the specimen between two gage marks. The modulus of elasticity, $E_s$, is determined by calculating the slope of the stress versus strain curve over the linear region.

The procedure in Appendix A9 of ASTM A 370 (1997) was used for the testing the mild steel bars. Figure 3.16 shows a specimen being tested. Three samples were randomly chosen from the No. 5 reinforcing steel bars used in the deck. The bars were cut to a length of 30 in. to fit into the Satec Systems UTM. A gage length of exactly 8 in. was punched at the center of each bar. Each bar was then positioned in the testing machine with a length of approximately 10.5 in. to 12 in. between the V-grips. The model SE2-50 2 in. gage length extensometer was then placed at the center of the specimen. A procedure was created for the Nuvision II Software to automatically load the specimens in tension at a rate of 30,000 psi/minute until the approximate yield point, then switch to a position controlled loading that would raise the tension head at a rate of 0.5 in./minute. To avoid damage during rupture, the extensometer was removed after yielding occurred. Once the specimen ruptured in tension, the software saved the load, strain, and position of the testing head at 0.8 second increments. The specimen was removed and fitted together so that the new measurement between the gage lengths could be determined for the elongation calculation.

3.5.4 Prestressing Steel Testing

The purpose of the prestressing steel testing was to determine the actual properties for a sample of the prestressing steel used for the fabrication of the six girders. Similar to the mild steel, the testing was done in accordance with the requirements of ASTM A 370

Figure 3.16 – Extensometer in place on Tensile Test in Satec UTM
(1997). The maximum tensile strength, \( F_{pu} \), and the modulus of elasticity, \( E_p \), are determined in the same manner as is used for the mild steel. Since the yield strength of prestressing steel is not as distinct as the yield strength of mild reinforcing steel, the procedure in Appendix A7 of ASTM A 370 (1997) was used for determining the yield strength.

The procedure for testing the prestressing steel was located in Appendix A7 of ASTM A 370 (1997). Three specimens of the ½ in. diameter steel used in the fabrication of the beams were tested. Since a 24 in. extensometer was not available for testing, the exact procedures of the ASTM could not be followed and elongation values were not obtained. The three specimens were cut to 40 in., which allowed for a 20 in. length between the V-grips. The ends of the strands were wrapped with aluminum foil to aid in gripping. The strands were then placed in the testing machine and the model SE2-50 2 in. gage length extensometer was installed at the beginning of each test. The NuVision II procedure was modified to load the specimens in tension at a rate of 40,000 psi/minute until the approximate yielding occurred, then switch to a position controlled loading that raised the tension head at a rate of 0.5 in./minute. To avoid damage, the extensometer was removed after yielding occurred. Once the specimen ruptured in tension, the software saved the data.

The specifications recommended placing an extensometer on the specimen when the load in the specimen was approximately 10 percent of the minimum specified breaking strength (0.10 x 270 ksi = 27.0 ksi) and adjusting the extensometer to a strain reading of 0.001 in./in. of the gage length. The specifications then required the load to be increased until the extensometer indicated an elongation of 1 percent had occurred. The yield strength would then be considered the stress at this reading. Since the extensometer attached to the Satec System UTM could not be zeroed during testing, it was placed prior to the start of testing and the strain reading at the specified stress of 27.0 ksi was recorded. The stress at which the strain increased to a value of 0.01 in./in. plus the strain reading at 27.0 ksi was then recorded as the yield stress.

3.6 Measurement Techniques

3.6.1 Sign Conventions

The following sign conventions in Figure 3.17 are used throughout this document. Strain gages were wired to produce negative strain when under compression. This can be thought of as a decrease in length as being negative. The dial gages read opposite the wirepots and the barrels
(LVDTs). For bridge design, most consider positive moment to cause compression in the top of the member and tension in the bottom of the member.

3.6.2 Load Cells

In order to measure the changes in reactions during the Static Phase, load cells were used under the elastomeric bearings and steel bearing plates. Since the readings were taken over a period of up to a month, it was decided to measure the reactions with a P-3500 Strain Indicator hooked up to two Switch and Balance Units. Each Switch and Balance unit allows for reading 10 channels; therefore, a total of 20 channels could be read with the system. Originally, an attempt was made to fabricate load cells specifically designed to measure the reactions while

Figure 3.17 – Sign Conventions
providing good overall stability for the system. It was found that these fabricated load cells were not accurate enough, and a combination of the fabricated load cells and conventional factory made transducer load cells were used.

The typical method used to calibrate or to check the calibration of a transducer load cell is to determine the sensitivity of the transducer for the maximum load reading that the gage will be used, also known as the full scale value. This is done by hooking up the transducer load cell to a strain indicator, such as the P-3500 Strain Indicator, as a full bridge transducer. The load cell is then placed in a machine and taken to a known load, say of 10,000 lb. The gage factor is then adjusted so that the reading on the strain indicator is some factor of 10. Depending on the sensitivity of the load cell, the reading could be 10,000 or 1,000 or 100 or 10. The relationship between the gage factor (GF), the sensitivity (S) in mV/V, and the full scale value (FS) is given by:

\[ GF = \frac{S4000}{FS} \]  

This method of calibrating transducer load cells works well for load cells used alone or used along with computer software that allows for inputting the sensitivity and the full scale value for each load cell individually. For this project, several different types of load cells along with up to 14 one-quarter bridge strain gages were attached to one strain indicator. Changing the gage factor on the strain indicator prior to taking every reading was determined to be impractical. Therefore, an alternate method for calibrating the load cells was used. A single gage factor of 2.00 was set on the strain indicator, and each load cell was loaded up to the maximum anticipated load, recording the actual load and the reading on the strain indicator at predetermined intervals. The results were then plotted and a calibration equation was determined for each load cell.

Since the test specimens have such large bearing areas (10 in. long by 24 in. wide bearing pads), insuring that the full depth test specimens would remain stable during testing was one of the main challenges. A single load cell under each bearing would not provide much stability at all, requiring that the specimen be heavily braced during testing. Since this heavy bracing may influence the results of the tests, it was decided to fabricate a more stable load cell to monitor reactions. The Four Column Load Cell was designed to help provide good stability during
testing and to enable long term monitoring of the reactions with the P3500 strain indicator unit.

Several constraints were considered in the design of the device. First, it needed to be sensitive enough to measure changes in the reactions during Static Phase testing and be able to read higher reactions during the remaining phases. To increase the sensitivity during the lower reactions of about 9 to 12 kips, the smallest total area of the four columns is desired. However, the total area must also be large enough to withstand loads of up to approximately 100 kips. In order to use conventional strain gages in a full-bridge, or Wheatstone bridge, circuit, the maximum total strain should be kept below about 1000 to 1500 micro-strain. Also, the structural stability of the columns due to buckling has to be considered.

A total of four Four Column Load Cells were fabricated as shown in Figure 3.18. It was found that using columns made up of TS 2x2x3/16 sections could satisfy all of the requirements. Structurally, the columns were checked for local buckling and found to be adequate. Based on an Fy of 46 ksi, the four columns are capable of safely supporting 122.9 kips at a stress level of 24.2 ksi. This maximum limit was determined using the AISC LRFD provisions. (Manual 2001)

The Wheatstone bridge circuit was determined to be best suited for determining the axial loads in the columns. Figure 3.19 shows the basic Wheatstone bridge configuration, the modified configuration with two gages used in series, and the actual wiring diagram used for the Four Column Load Cell. A V indicates a vertical gage and an H indicates a horizontal gage. The conventional notation for positive power (\(P^+\)), negative power (\(P^-\)), positive signal (\(S^+\)), and negative signal (\(S^-\)) is used. Also, the
conventional wiring colors (R)ed, (W)hite, (G)reen, and (B)lack are used. This circuit can be made to compensate for temperature by using similar gages in the opposite and adjacent arms of the circuit. A total of eight strain gages, one vertical and one horizontal on each of the columns were used. This arrangement was wired in an attempt to account for eccentricity due to off center loading (Hannah, 1992). Using this arrangement required only one four-stranded wire for reading each load cell. Therefore, all four load cells could be attached to one switch and balance unit at the same time, leaving six channels free.

![Wheatstone Bridge Wiring Diagram](image)

Figure 3.19 – Wheatstone Bridge Wiring Diagram

The sensitivity of the device to loading can be theoretically determined by the following:

\[
\frac{V_o}{V_i} = \frac{F \epsilon_1 (1 + \nu)}{2 + F \epsilon_1 (1 - \nu)}
\]

(3.5)

where \(V_o\) is the output or signal voltage, \(V_i\) is the input or power voltage, \(F\) is the gage factor (usually around 2.0), \(\epsilon_1\) is the strain in the member, and \(\nu\) is the Poisson’s ratio (about 0.3 for steel). The strain in the system is given by:

\[
\epsilon_1 = \frac{P}{AE}
\]

(3.6)

where for the four columns, the total area \(A\) is 5.08 in.\(^2\) and \(E\) is 29(10\(^6\)) psi. Therefore, for a P
of 100 pounds, the strain, $\varepsilon_1$, is 0.679 micro-strain. Using an input voltage, $V_i$, of 2 Volts DC, and a gage factor $F$ of approximately 2.0, the output voltage, $V_o$, can be calculated to be about 1.20 micro-volts per 100 pounds of load. This means that a system that can be read to the nearest micro-volt, could distinguish load changes of $100 \times (1/1.20) = 83$ pounds. An input voltage of 10 Volts DC improves the sensitive by 5 times, or to about every 17 pounds.

### 3.6.3 Vibrating Wire Gages

Geokon Model 4200 vibrating wire gages (VWG) were chosen for installation in the girder specimens and at the center of the diaphragm connection for Tests 1 and 2. Figure 3.20 shows the model 4200 gage attached to a number 3 bar with wood spacer blocks and ties ready for installation in the girder. These gages were chosen because of their ability to provide reliable readings of strain when embedded in concrete. The gages are constructed of a hollow aluminum tube with a wire attached at each end to an enlarged disk. When cast in concrete, the ends of the gage will move simultaneously with the concrete. A small magnet at the center of the gage is used to excite the wire and the resonant frequency of the vibrating wire is then read with an electromagnetic coil. The resonant frequency of the wire ($f$) is a function of the tension ($F$), length ($L_w$), and mass of the wire ($m$), as shown.

$$ f = \frac{1}{2L_w} \sqrt{\frac{F}{m}} $$  \hspace{1cm} (3.7)

Using the fact that the force ($F$) is equal to the product of the strain, Young’s Modulus, and the area of the wire, and manipulating the equation with the actual gage lengths and the density of the wire, the strain ($\varepsilon$) in microstrain can be determined as a function of the frequency ($f$) as in the following:

$$ \varepsilon = 3.304 \times 10^{-3} f^2 $$  \hspace{1cm} (3.8)

During the Fabrication Phase, the VWGs were read using the CR23X Micrologger. A nominal batch factor, $K$, equal to 0.958 was supplied by the manufacture of the gages in order to account for differences in batches. This factor is included in the program written for the CR23X. For readings taken with the Geokon Model GK-403 Vibrating Wire Readout box, all readings must be multiplied by this factor.

Since the wire used inside of the gages is made out of metal, the coefficient of thermal
expansion of the wire and the surrounding concrete are different. The manufacture recommends correcting for this by adding a term to correct the actual (apparent) reading. Each gage has a thermistor which records the gage temperature in °C.

The vibrating wire gages measure changes in compression as a negative number and changes in tension as positive number. The manufacturer recommends using a value for the coefficient of thermal expansion for the wire, \( C_1 \), of 12.2 microstrain/°C (6.78 microstrain/°F), and for the concrete, \( C_2 \), of 10.4 microstrain/°C (5.78 microstrain/°F). If the temperature changes from an initial reading \( T_0 \) to a final reading \( T_1 \), the correction for temperature is given as:

\[
(T_1 - T_0)(C_1 - C_2)
\]  
(3.9)

To better understand this correction, an example situation is provided. Assume that a block of concrete 6 in. long has a vibrating wire gage wire installed so that the ends of the wire are attached to the ends of the concrete. Also assume that the coefficients of expansion for the steel wire and concrete, \( C_1 \) and \( C_2 \), are 12.2 and 10.4 microstrain/°C, respectively. As shown in Part “A” of Figure 3.21, the wire and the concrete are the exact same length at a given temperature, \( T_0 = 0 \)°C. Now, allow the concrete to undergo an increase in temperature to temperature \( T_1 = 100 \)°C. As shown in Part “B”, the concrete will expand to a final length of 6.00624 in. (6+0.0000104x100x6). However, the wire wants to expand to a final length of 6.00732 in. (6+0.0000122x100x6). Since the wire is attached to the ends of the concrete, forces \( F \) are created at the ends of the wire that put the wire into compression. These forces cause a compressive strain in the wire equal to the desired change in length of the wire (0.00108 in.) divided by the length of the wire at the given temperature (6.00732 in.). This strain, equal to approximately 180 \( \mu \varepsilon \), would cause the gage to record a strain change of negative 180 \( \mu \varepsilon \), or a compressive strain of 180 \( \mu \varepsilon \). Since no loads or restraints are applied to the concrete block, the
actual change in strain should be zero. Therefore, the correction for temperature must be added to give the actual change in strain of the concrete. This correction, which is \((T_1-T_0)(C_1-C_2)\) is equal to \((100-0)(0.0000122-0.0000104)=180 \mu \varepsilon\). When added to the reading of -180 \(\mu \varepsilon\), the resulting change in strain is zero as expected.

One 6 in. by 12 in. control cylinder was cast during the fabrication phase with a vibrating wire gage located in the center of the specimen along the vertical axis of the cylinder. Figure 3.22 shows the control cylinder prior to casting the concrete.

When the recommended correction for temperature is made, the change in strain for a specimen that undergoes only thermal movements should be zero. To confirm that this is actually the case, two experiments were conducted to test the ability of the vibrating wire gages to correctly compensate for changes in temperature. The purpose of the first experiment was to determine the actual coefficient of thermal expansion for the concrete being used. Four cylinders, one from
deck 1, one from deck 2, and two from the girders, were instrumented with Demec points at a 6 in. gage length. The cylinders were then placed in an oven at 110 °C for 20 hours. The cylinders were then allowed to cool to room temperature overnight, and a series of readings at different temperatures were made using the Whittemore Gage.

The purpose of the second experiment was to confirm the temperature correction formula of the vibrating wire gages. The strain reading of the control cylinder with the vibrating wire gage was recorded at room temperature. The cylinder was then subjected to different temperatures by heating in an oven and cooling in a refrigerator and/or a freezer. Once the cylinder reached the equilibrium temperature, the strain reading and the thermistors were recorded. To confirm the operation of the gage, the control cylinder was also loaded in the Satec UTM and strain readings were recorded at different load levels.

3.6.4 Electrical Resistance Gages

Conventional electrical resistance gages were used on the longitudinal reinforcing in the deck for all tests and on a stirrup located near the bottom of the continuity connection for Tests #1 and #2. For the Static Phase, the gages were recorded with the P-3500 Strain Indicator and the Switch and Balance Units. For the Service and Cyclic Phase, the gages were recorded with the StrainSmart software.

For the Four Column Load Cells, 120 ohm gages were used. For all gages used on the reinforcing steel, 350 ohm gages were used. The exact type and sizes of the gages are shown in Table 3.2.

The following procedure was used for installing and wiring the gages. This procedure worked well, with only one gage out of over 70 gages found not to work properly. Using this convention and hooking up the gages to the P-3500 Strain Indicator as recommended will produce readings with the conventional sign convention, compression is negative and tension is positive.

Prepare the Bar:

1. Use hand grinders to prepare an area on the bar slightly larger than the gage to be applied
2. Use the pneumatic sander to finish the surface to a nearly flat surface
3. Spray with degreaser and wipe dry
Apply the Gage:

1. Wipe the surface of the bar and an empty gage box with alcohol (at least 91%)
2. Place gage on box and pick up with a piece of tape (the ends folded over)
3. Tape gage to bar, and partly pull back gage so that gage is perpendicular to the bar
4. Wipe excess blue prep off of the brush by hitting against the inside top of the container ten times
5. Apply the blue prep to the bar and the underside of the gage and wait one minute
6. Apply 2 drops of adhesive and press down on gage with thumb for two minutes
7. If possible, wait overnight before proceeding

Wire the Gage:

1. Wipe the terminals of the gage with a typewriter eraser to produce a shine
2. Apply flux and solder a bead to the terminals
3. Place a small piece of butyl rubber (sticky type) to end of terminal to hold wires in place
4. Twist the black and white wires together. Apply flux and solder to both wires
5. Cut both the black/white twisted wire and the red wire so that only about 1/16 in. extends
6. Press wires into black sticky rubber and place a wire tie around bar and wire
7. Apply flux and solder the wires down to the gage terminals, black/white to one side, red to the other side
8. Create a loop in the wire for stress relief and loosely wire tie beyond the black rubber

Waterproof the Gage:

1. Apply several coats of white coating, m-coat D, to gage and terminals, allowing at least two hours between coats, allow to dry overnight
2. Cover the gage with butyl rubber (the sticky type)
3. Cover the gage with the neoprene pad (non-sticky type) and fully tape around the bar
4. Seal the edges of the tape with m-coat B, or use Rubberize-It, the material for coating tool handles

3.6.5 Displacements

During the Static and Cyclic Phase, four methods were used to monitor deflections of the test specimen. Three of the methods used the StrainSmart software and the System 5000 to monitor the gages. For the first method, end deflections were monitored with Celesco Position Transducer wirepots. The figure in Section 4.4 Load Cell Calibration and Testing shows a wirepot. The wirepots have a 10 in. measurement range and were attached to the underside of the ends of the girder with magnets. The gages were entered into the software as Transducers and excited with 2 volts. They were manually calibrated with the digital counter to improve the sensitivity of the readings. The practical sensitivity was found to be approximately +/- 0.005 in.
The second method used Trans-Tek series 350 gaging transducers, commonly called LVDTs or barrels, to measure the crack openings at the bottom of the girder and diaphragm interface for Test 1 and Test 2, and to measure relative girder end movements at the bottom of the girders and near the top of the web for Test 3. The barrels were entered as High Level instruments in the software, instead of as LVDTs. The barrels were excited with 10 volts and a gain of 1 and were also manually calibrated with the digital counter. The practical sensitivity was found to be approximately +/- 0.001 in.

The third method used the output from the MTS 458.10 MicroConsole to give the actual active end displacement as determined by the MTS loading ram. The end displacement and the MTS load were entered into the software as High Level devices. The manual calibration for the deflection was entered as -10,000 mV = -10 in., and +10,000 mV = +10 in. For the load, the manual calibration was -10,000 mV = -200,000 lbs, and +10,000 mV = +200,000 lbs.

To confirm the electronic measuring devices, the fourth method used conventional dial gages capable of reading 0.001 in. Under the girder, a dial gage with a 2 in. stroke was used at the active end and gages with a 1 in. stroke were used at the other locations. Dial gages were also placed at the end of girder and diaphragm interface for Test 2 and at the ends of the girders at the center for Test 3.

3.6.6 Thermocouples

Type TT-T-20-SLE thermocouple wire using a positive blue wire (copper) and a negative red wire (constantan) was used. The ends of the wire were twisted together and soldered with a silver solder to produce the thermocouple. In order to attach to the HH21 microprocessor, type NMP mini connectors were attached to the wire allowing the ends to be plugged into the
3.6.7 Elastomeric Bearing Pads

The elastomeric bearing pads used for the testing were designed by VDOT for a 2 span continuous structure, each span being 80 ft with PCBT 45 girders at 6 ft on center with a 7 ½ in. deck and an HS 20-44 Live Load. The non-composite dead load was 1541 plf (pounds per linear foot) and the composite dead load was 270 plf. Nine possible bearing sizes were sent to Scougal Rubber and the final 1.9375x10x24 laminated pad with 2-11 gage shims was chosen. One of the bearing pads was placed in the Satec UTM and loaded up to 80 kips. Since an extensometer could not be attached to the pad, the change in position of the machine was used to estimate the change in position of the pad. Doing this slightly overestimates the change in position of the pad, because the testing frame will deflect under loading. The data showed a slightly curved relationship between load and position. However, fitting the data to a straight line still produced a relationship with a high R-squared value of 0.98. Using the data from the test, the stiffness of the pad was estimated to be approximately 904 kips/in. and the modulus of elasticity of the pad was estimated to be approximately 7.3 ksi.
4 Laboratory Results and Discussions

4.1 Concrete Testing Overview

Figure 4.1 shows the averaged measured compressive strength and the modulus of elasticity/1000 in ksi for the concrete in the girders. The concrete for the girders gained strength rapidly, reaching the specified 28 day strength of 8 ksi at an age of approximately 3 days. Also, the modulus of elasticity reached the predicted value of 57 times the square root of the actual compressive strength by the time of the release of strands at 2.1 days. Because of the shortage of Type III cement, Type II cement was used along with several new admixtures. The concrete had good workability when first mixed, but became stiff and unworkable in a matter of about 15 minutes. It set so fast that one batch set in a truck and had to be discarded. Making the cylinders proved to be difficult, and it is possible that some were not completely consolidated because of the mix becoming stiff so quickly. Multiple tests were made at a given time and all data points are presented as averaged values. For compressive strength and modulus of elasticity tests, each data point is the average of 2 to 4 tests. For the split cylinder and the flexural beam tests, each data point is usually the average of 2 tests. A summary of the concrete test results is included in Appendix E.

![Figure 4.1 – Girder Concrete Compressive Strength and Modulus Results](image-url)
Figure 4.2 shows the results of the split cylinder and the flexural beam testing for the girder concrete. Typically, the flexural beam tests produce results as much as 20-50 percent higher than the split cylinder tests (MacGregor, 1997). However, none of the tests were this much higher. For tests performed at 5, 28, and 222 days, the flexural beam tests were 1.14, 0.95, and 1.04 times the split cylinder tests. The values for both of the tests remained fairly constant over the course of testing. Split cylinder testing has become more common because it is easier.

![Figure 4.2 - Girder Concrete Split Cylinder and Flexural Beam Results](image)

Figure 4.3 shows the development of the compressive strength, the modulus of elasticity, and the split tensile strength for each of the separate deck concretes. The same mix was ordered for all three tests and the slumps at the time of placing the decks were all within 5 in. +/- 0.5 in. The concrete for Tests 1 and 2 developed similar compressive and split cylinder strengths throughout the testing. The modulus of elasticity of the concrete in Test 2 developed slower than it did in Test 1, but at approximately 28 days, reached the same value. Generally, all properties for the concrete in Test 3 developed slower and were lower at a given concrete age than the same properties for Tests 1 and 2. Different temperatures may have contributed to the performance of the concrete in Test 3. But, since all tests were performed inside the Structures and Materials
Figure 4.3 – Deck Concrete Test Results
Laboratory, where the temperature is controlled by thermostats, this is unlikely. It is more likely that although the same mix was ordered, a slightly different mix was provided. The average of two tests was used for each data point, and the data points are connected to show the trend.

### 4.1.1 Tensile Strength Testing

The ACI Building Code (2002) defines the modulus of rupture for normal weight concrete (the stress when first cracking will occur) as 7.5 times the square root of the compressive strength ($7.5(f'c)^{0.5}$) in Section 9.5.2.3 for use in determining deflections. In Section 11.4.2.1, it recommends a more conservative value of 6 times the square root of the compressive strength for strength calculations. The AASHTO Standard Specifications uses the same allowable prediction for the modulus of rupture (AASHTO 2002).

For the girder concrete, the actual splitting tensile strengths are plotted versus the actual compressive strengths in Figure 4.4. Also, the predicted value of $7.5(f'c)^{0.5}$ is plotted. For all values of compressive strength, the actual splitting tensile strength as estimated by the splitting tensile strength results are an average of 180 psi higher than predicted by the recommended equation of $7.5(f'c)^{0.5}$.

![Figure 4.4 – Splitting Tensile Strength of Girder Concrete](image-url)
Equations for the tensile strength as a function of the compressive strength were fitted to the data. The approach taken was to first attempt to find the best possible equation to describe the data and determine how well it describes the data by determining its R-squared value. Then, an attempt was made to describe the data with an equation in a form similar to what is typically used. Simply using \( 9.5(f'c)^{0.5} \) gives a good relationship for compressive strengths up to about 9,500 psi. The relationship shown of \( 5(f'c)^{0.5} + 425 \) gives an R-squared relationship of 0.40, comparable to the best relationship found but in a similar form to the relationship typically used.

For the deck concrete, as shown in Figure 4.5, the relationship \( 7.5(f'c)^{0.5} \) predicts the actual splitting tensile strength very well, producing an R-squared value of 0.91. Therefore, for predicting splitting tensile strength, the ACI recommendation works very well for the lower strength deck concrete, but underestimates the splitting tensile strength for the girder concrete.

![Figure 4.5 – Splitting Tensile Strength of Deck Concrete](image)

4.1.2 Modulus of Elasticity Testing

The ACI Building Code (2002) allows the modulus of elasticity \( (E_c) \) of normal weight concrete to be taken as 57,000 times the square root of the compressive strength \( (57,000(f'c)^{0.5}) \) in psi, AASHTO allows for the same prediction (AASHTO 2002). The code does not
specifically address the compressive strength of the concrete. For higher strength concrete, studies at Cornell University have indicated that the ACI equation overestimates that modulus of elasticity, and recommends using the equation $40,000(f'c)^{0.5} + 1,000,000$ to estimate the modulus of elasticity in psi (Nilson 1987).

For the higher strength girder concrete, a clear relationship between the compressive strength and the modulus of elasticity was not noticed. As shown in Figure 4.6, both the ACI and the Nilson recommended equations underestimate the actual modulus of elasticity for most compressive strengths under 9,500 psi. Surprisingly, the ACI equation does better than the Nilson equation, even though the Nilson equation is intended to be better for higher strength concretes. The best possible relationship was found to have an R-squared value of 0.32, and the best relationship shown in a form typically used has an R-squared value of 0.28.

For the deck concrete, a much better correlation between the compressive strength and the modulus of elasticity was noted, as shown in Figure 4.7. As expected, the ACI equation predicts the modulus better than the Nilson equation. Both equations slightly underestimated that actual modulus of elasticity, with the ACI equation producing an R-squared value of 0.73.

Figure 4.6 – Modulus of Elasticity of the Girder Concrete

For the deck concrete, a much better correlation between the compressive strength and the modulus of elasticity was noted, as shown in Figure 4.7. As expected, the ACI equation predicts the modulus better than the Nilson equation. Both equations slightly underestimated that actual modulus of elasticity, with the ACI equation producing an R-squared value of 0.73.
and the Nilson equation producing an R-squared value of 0.52. Simply increasing the constant in the ACI equation from 57 to 61 improved the R-squared value to 0.82.

![Figure 4.7 – Modulus of Elasticity of the Deck Concrete](image)

### 4.1.3 Shrinkage Testing

For Test 1 and Test 2, obtaining consistent shrinkage specimen readings with the Whittemore gage proved to be difficult. For Test 1, the recommended procedure involved zeroing the Whittemore gage prior to each use with a steel bar with a known 8 in. gage length. This procedure caused the zero of the gage to vary by as much as approximately +/- 0.0003 in. Although three ten-thousandths of an inch may not seem like much, a change of 0.0001 in. is equivalent to a change in shrinkage of 12.5 microstrain. For Test 2 and Test 3, the gage was only zeroed at the beginning of the testing, which seemed to decrease some of the error. At the time it seemed that the gage was working well, but too much variability was still found in the measurements. Prior to Test 3, the set screw of the gage was tightened so that the plunger would barely move in and out. This, along with using the Demec points instead of the Whittemore points, improved the repeatability of the gage. Therefore, for Test 3, the gage worked fairly well.
Figure 4.8 and Figure 4.9 show the results of the shrinkage tests for all three tests as well as the predicted shrinkage from ACI 209 and MC-90. Figure 4.8 shows the cumulative shrinkage strain from the time of the first reading while Figure 4.9 shows the cumulative shrinkage strain from the end of the moist curing which was at 7 days.

The results indicate that the MC-90 predicted shrinkage is much less at the early ages of the deck than actually observed. Since the specimens were cast indoors, an average relative humidity of 60 percent was used for the prediction equations. The sides of the specimens were sprayed with a sealant providing a volume to surface ratio of the specimens of three. Both the ACI-209 and MC-90 predict similar ultimate shrinkage values at 10,000 days, 521 and 505 microstrains respectively, but it is clear that at the early ages, the MC-90 predicts much lower shrinkage. The ACI-209 prediction is similar to the results of Test 1, both predicting approximately 200 microstrains of shrinkage at 28 days. Considering the shrinkage that occurred from the 7 day cure period, Test 2 displayed shrinkage of approximately 275 microstrains at 28 days while ACI 209 predicts 200 microstrains and MC-90 predicts only approximately 80 microstrains of shrinkage.
Test 3, unlike Test 2, showed positive shrinkage values for approximately the first 8 days. This indicates that expansion of the specimen occurred during this period. This is certainly possible since the results of the Static Phase tests showed that the end reactions first increase, indicating an expansion at the top of the composite section. However it is interesting that Test 2 did not show any signs of expansion during the early readings. Since both tests were performed indoors using the same mix design, the controlling parameters were similar, and similar results were expected. As mentioned earlier, it is possible that difficulty in obtaining the readings may have produced some of the variability. It is also possible that the actual shrinkage is sensitive to the mix and environmental conditions during the casting and early curing. There is not enough data to make any definitive conclusions about the shrinkage testing, but it is evident that the actual shrinkage that occurs during the early ages of curing may vary a great deal from batch to batch.

4.2 Mild Steel and Prestressing Steel Testing

Prior to testing the mild steel and prestressing steel specimens, the calibration for the SE2-50 2 in. extensometer was checked using the digital counter. It was found that the
extensometer was providing strain readings that were larger than the actual strain being introduced. Therefore, the actual strains for all the tests were reduced by a correction factor of 0.952.

### 4.2.1 Mild Steel Testing

Three samples of the No. 5 reinforcing bars used for the deck reinforcing were tested. The results of these tests are shown in Table 4.1 and Figure 4.10 through Figure 4.12. All of the specimens displayed a distinct linear region after the V-grips had settled into place and firmly gripped the specimens. The modulus of elasticity was calculated over the linear range of the stress versus the strain plot for a stress from 15 ksi to 55 ksi by determining the best fit line over this region. The slope of this line is the modulus of elasticity. The relationship displayed excellent correlation, with R-squared values of 0.999 for each test.

<table>
<thead>
<tr>
<th>Bar Mark</th>
<th>Yield Strength ksi</th>
<th>Tensile Strength ksi</th>
<th>Modulus of Elasticity ksi</th>
<th>Percent Elongation %</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>61.5</td>
<td>99.5</td>
<td>27,500</td>
<td>14</td>
</tr>
<tr>
<td>B</td>
<td>64.5</td>
<td>102.5</td>
<td>28,500</td>
<td>14</td>
</tr>
<tr>
<td>C</td>
<td>62.0</td>
<td>100.0</td>
<td>27,000</td>
<td>14</td>
</tr>
<tr>
<td>Average</td>
<td>62.7</td>
<td>100.7</td>
<td>27,667</td>
<td>14</td>
</tr>
<tr>
<td>Min. Required</td>
<td>60.0</td>
<td>90.0</td>
<td>N/A</td>
<td>9</td>
</tr>
</tbody>
</table>

The tensile strength, yield strength, and elongation values all met the requirements of steel usually used in decks, ASTM A 615 (1996). The average modulus of elasticity was determined to be 27,700 ksi, which is slightly lower than the commonly used value of 29,000 ksi.

### 4.2.2 Prestressing Steel Testing

Three samples of the ½ in. diameter grade 270 prestressing strands used for fabrication of the girders were tested. The results of these tests are shown in Table 4.2 and Figure 4.13 through Figure 4.15. All of the specimens displayed a distinct linear region after the V-grips had settled into place and firmly gripped the specimens. The modulus of elasticity was calculated over the linear range of the stress versus the strain plot for a stress from 50 ksi to 200 ksi by determining the best fit line over this region. The slope of this line is the modulus of elasticity. The relationship displayed excellent correlation, with R-squared values of 0.999 for each test. The
Figure 4.10 – Mild Steel Bar “A” Tensile Test Results

Figure 4.11 - Mild Steel Bar “B” Tensile Test Results
extensometer initially slipped during the testing of Bar “C”. Therefore, the strain data for this test was adjusted to account for the initial slip.

The tensile strength, yield strength, and elongation values all met the requirements of strands usually used in girders, ASTM A 416 (1999). The average modulus of elasticity was determined to be 29,200 ksi, which is slightly higher than the commonly used range of 27,000 ksi to 28,500 ksi.

Table 4.2 – Summary of Prestressing Steel Properties

<table>
<thead>
<tr>
<th>Strand Mark</th>
<th>Yield Strength ksi</th>
<th>Tensile Strength ksi</th>
<th>Modulus of Elasticity ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>255.3</td>
<td>274.9</td>
<td>29,000</td>
</tr>
<tr>
<td>B</td>
<td>256.6</td>
<td>281.6</td>
<td>29,500</td>
</tr>
<tr>
<td>C</td>
<td>256.9</td>
<td>275.0</td>
<td>29,000</td>
</tr>
<tr>
<td>Average</td>
<td>256.3</td>
<td>277.2</td>
<td>29,167</td>
</tr>
<tr>
<td>Min. Required</td>
<td>242.9</td>
<td>269.9</td>
<td>N/A</td>
</tr>
</tbody>
</table>
Figure 4.13 – Prestressing Strand “A” Tensile Test Results

Figure 4.14 – Prestressing Strand “B” Tensile Test Results
The experiment performed to determine the coefficient of thermal expansion of the deck and girder concrete gave good results. In order to calculate the coefficient of thermal expansion, the actual Whittemore readings were divided by the original gage length of 6 in. Since the Whittemore gage reads expansion in a member as negative, this value was then multiplied by negative one and plotted versus the temperature. The slope of this line is the coefficient of thermal expansion. Figure 4.16 shows the best fit lines (with the y-intercept removed) and the corresponding slopes for the four cylinders. For the deck concrete, a coefficient of thermal expansion of 5.1 $\mu$e/°F (9.2 $\mu$e/°C) was determined. For the girder concrete, a slightly lower coefficient of thermal expansion of 4.7 $\mu$e/°F (8.5 $\mu$e/°C) was determined. For both cases, the actual calculated values were lower than the vibrating wire gage manufacture’s recommended value of 5.8 $\mu$e/°F (10.4 $\mu$e/°C). This makes sense, because, as per a phone conversation with the manufacturer, the recommended value of 5.8 $\mu$e/°F (10.4 $\mu$e/°C) is intended for normal strength concrete with normal aggregate percentages. The fabricator indicated that the stone used for manufacturing the girders was granite. According to the PCI Design Manual, a concrete made
from this type of stone could have a coefficient of thermal expansion as low as 3.8 με/°F (6.84 με/°C). (PCI 1997)

Two additional experiments were performed on the larger 6 in. by 12 in. control cylinders made from the girder concrete. Both of these two experiments produced higher values for the coefficient of thermal expansion of the girder concrete, 5.5 με/°F (9.9 με/°C) and 5.7 με/°F (10.3 με/°C). It is possible that water inside the control cylinder may have caused the readings to be higher. Since water has a volumetric coefficient of thermal expansion of approximately 116.7 με/°F (210 με/°C), the approximate linear coefficient of thermal expansion can be estimated to be 1/3 this value, 38.9 με/°F (70 με/°C). Therefore, the presence of a small amount of water within the cylinder may cause the weighted average of the coefficient of thermal expansion to be higher (Contemporary 1990).

The actual measured value of the coefficient of thermal expansion for the girder concrete was used to confirm the temperature correction of the vibrating wire gage in the control cylinder. Since the cylinder was subjected to only temperature changes, the expected reading of strain after the temperature correction should always be zero. Three separate tests were performed
with the control cylinder. The results of the tests were mixed. Table 4.3 shows the actual readings at the given temperatures. The apparent change is the difference from the reading and the reading at the lowest temperature. The corrected change is the difference from the reading and the reading at the lowest temperature with the correction for the change in temperature using the experimentally determined value for the coefficient of thermal expansion of $4.7 \, \mu \varepsilon /^{\circ}F$ ($8.5 \, \mu \varepsilon /^{\circ}C$). This value should be approximately zero for all readings. The next column shows the calculated value of the coefficient of thermal expansion of the concrete required to make the corrected reading equal to zero.

Table 4.3 – Control Cylinder VWG Readings

<table>
<thead>
<tr>
<th></th>
<th>Temperature °C</th>
<th>Reading με</th>
<th>Apparent Change με</th>
<th>Corrected Change με</th>
<th>Solve for $\alpha$ με/°F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
<td>-21.5</td>
<td>2466</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>-15.2</td>
<td>2444</td>
<td>-22</td>
<td>2</td>
<td>4.84</td>
</tr>
<tr>
<td></td>
<td>19.5</td>
<td>2388</td>
<td>-78</td>
<td>75</td>
<td>5.72</td>
</tr>
<tr>
<td></td>
<td>22</td>
<td>2402</td>
<td>-64</td>
<td>99</td>
<td>5.96</td>
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<tr>
<td></td>
<td>59.4</td>
<td>2375</td>
<td>-91</td>
<td>212</td>
<td>6.15</td>
</tr>
<tr>
<td>Test 2</td>
<td>-4.1</td>
<td>2393</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>18.2</td>
<td>2338</td>
<td>-55</td>
<td>28</td>
<td>5.41</td>
</tr>
<tr>
<td></td>
<td>19.8</td>
<td>2354</td>
<td>-39</td>
<td>50</td>
<td>5.87</td>
</tr>
<tr>
<td></td>
<td>21.8</td>
<td>2341</td>
<td>-52</td>
<td>45</td>
<td>5.66</td>
</tr>
<tr>
<td></td>
<td>50.3</td>
<td>2340</td>
<td>-53</td>
<td>150</td>
<td>6.24</td>
</tr>
<tr>
<td>Test 3</td>
<td>-21.4</td>
<td>2446</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>18.2</td>
<td>2338</td>
<td>-108</td>
<td>40</td>
<td>5.26</td>
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<td>50.2</td>
<td>2320</td>
<td>-126</td>
<td>142</td>
<td>5.80</td>
</tr>
</tbody>
</table>

From the experiment, it can be observed that, for this one cylinder, the correction for temperature is sensitive to the actual value of the coefficient of thermal expansion of the concrete. For all three tests, the corrected readings over the range of temperatures investigated indicate that an error of approximately $0.23 \, \mu \varepsilon /^{\circ}F$ ($0.41 \, \mu \varepsilon /^{\circ}C$) was observed. For temperatures that remain fairly constant, this error is not too significant. However, for temperatures that change a great deal, such as observed during the fabrication phase, this error can be significant. It can also be observed that the required coefficient of thermal expansion of the concrete to cause the corrected value to be zero is always greater than the experimentally measured value and also
increases with increasing temperature. A value of 6.24 με/ºF (11.23 με/ºC) as observed in the highest reading in Test 2 nearly approaches the recommended value for steel, which is 6.8 με/ºF (12.2 με/ºC).

Unfortunately, only one control cylinder with a vibrating wire gage was made during the fabrication phase. A larger number of control cylinders would be needed to make more definitive conclusions about the temperature correction of the vibrating wire gages. In taking the readings, the control cylinder was allowed to remain at the given temperature usually overnight, for approximately 20 hours. The shortest duration used in the freezer was approximately 8 hours. Readings for the thermistor inside the gage and a thermocouple placed inside the freezer were in good agreement, indicating that the cylinder had reached an equilibrium temperature. Once placed in the oven, the coating on one of the four wires, the red wire, began to degrade. Eventually, the wire became exposed and had to be cut and re-stripped. Although all readings taken were stable at the time, it is possible, but unlikely, that placing the wire in the oven could have adversely affected the readings.

4.4 Load Cell Calibration and Testing

The factory made load cells were calibrated with the P-3500 Strain Indicator using a gage factor of 2.000 prior to each of the three tests. Table 4.4 shows the calibration equations used for the four factory made load cells and the equations used for the Four Column Stools.

The factory made load cells worked well for recording the reactions during the Static Phase. All of the load cells were balanced to a reading of zero prior to use, and returned to approximately zero after use. This indicates that drift of the load cells was not a significant factor. In order to improve the overall stability of the test setup, two load cells were placed at both the passive end (Reaction R1) and the active end (Reaction R4) of the girders. At the center, the Four Column Load Cells were used at Reaction R2 and Reaction R3. Figure 4.17 shows a completed Four Column Load Cell in place at the passive end of the test. From top to bottom starting at the bottom of the girder is: the elastomeric bearing, a 1 in. plate, two 50 kip load cells, the four column load cell, a wire pot, and the leveling frame containing Rockite
The Four Column Stools did not perform as well as expected. The first stool was fabricated and calibrated in the Satec UTM and seemed to work well. The relationship of load to the reading of the P-3500 Strain Indicator was not completely linear as expected, but was repeatable. The other three stools were fabricated and prepared for use. When the first girder was set on the stools, the reading was only about half of what it should have been. A single load cell was placed and confirmed that the end reaction of the girder alone was about 6 kips. Then, two load cells placed 16 in. apart under a single end reaction indicated that one load cell read 4 kips while the other read 2 kips.

The stool had been tested with off center loading and found to work acceptably. However, it was found that it would not compensate for the actual loading condition where the loading on one side of the girder end was much higher than on the other side. An attempt was made to completely level the stools to prevent the difference in loading, but this did not help. Since the girders were so short and stiff, irregularities in the fabrication process caused unevenness in the end reactions. The girders were slightly twisted, and the end reaction was higher on opposite sides from one end to the other as expected for a slightly twisted member.

It was found that the averaging effect of the strain gages placed in series caused the stools
not to function properly. In the design of the stools, a literature review found that most sources stated that “…strain gages placed in series in a given arm of a Wheatstone bridge are additive.” This statement is partly true, partly false, and somewhat misleading. Strain gages that are placed in series actually produce an output which is an average of the two gages instead of the sum of the two gages. The basic property for a strain gage that enables it to be used as a load determining device is the fact that the gage factor, GF, stays constant over a large range. This relationship is:

\[
GF = \frac{\frac{\Delta R}{R}}{\frac{\Delta L}{L}} = \frac{R}{\varepsilon} \quad (4.1)
\]

where \(\frac{\Delta R}{R}\) is the change in resistance over the original resistance and \(\frac{\Delta L}{L}\) is the change in length over the original length which is equal to the strain, \(\varepsilon\). As an example, assume that two 120 ohm gages are placed in series in a given arm of a Wheatstone bridge, and one undergoes a strain that produces a \(\Delta R\) of 2 units while the other undergoes a strain that produces a \(\Delta R\) of 4 units. Since the two gages are in series, one may expect that the total output of the two gages to be \(\frac{\Delta R}{R} + \frac{\Delta R}{R} = 2/120 + 4/120 = 6/120=0.05\); however, this is not correct. Since the two are in series, the total resistance of the two is additive, and the total resistance \(R = 120 + 120 =240\). Therefore, the actual output is \((\Delta R + \Delta R)/R = (2+4)/240 = 6/240=0.025\). In other words, the readings average instead of simply adding together.

Since the end reactions of the girders were so much higher on one side than the other, the reading of 4 kips on one side and 2 kips on the other averaged to give a reading of \((4+2)/2 = 3\) kips instead of the total 6 kips. The stools were still used in the test setup and were monitored throughout all of the tests. The results help to confirm that when the outside reactions were increasing, that the inside reactions were decreasing, and vice versa. At higher loads, the unevenness in the end reactions became less prevalent, and the stools gave better readings. Since the actual values are not correct, the data for the stools is used only to confirm changes in reactions.
5  Fabrication Phase Results and Discussions

5.1  Overview of Steam Curing

Five thermocouples were installed in each girder as shown in Figure 5.1. Figure 5.2 shows the location of the vibrating wire gages. Seven of the thermocouples in the girders and two additional ones within the prestressing bed were monitored and recorded at ten minute intervals with the Campbell CR23X during the fabrication of the girders. The remaining thermocouples were recorded periodically with the hand held HH21 reader. In addition to the thermocouples, the 31 thermistors within the vibrating wire gages were monitored and recorded.

![Figure 5.1 – Thermocouple and Vibrating Wire Gage Locations](image)

According to both the Standard and LRFD AASHTO Specifications, application of the heat from the steam curing shall “…increase at an average rate not exceeding 40°F per hour until curing temperature is reached.” Also, the “…maximum curing temperature within the enclosure shall not exceed 160°F.” The steam shall be turned off is such a way so that bed temperature shall “…not decrease at a rate to exceed 40°F per hour until a temperature 20°F above the temperature of the air to which the concrete will be exposed has been reached (AASHTO 2001, 2002).” The PCI Bridge Design Manual indicates that some codes allow a maximum concrete temperature of 190°F and that some producers allow temperatures to rise to 200°F. (PCI
1997) The Bayshore Steam Schedule Sheet shows a maximum allowable rise of 40°F per hour and that the temperature in the bed is to be held between 145°F and 150°F.

### 5.1.1 Temperature Readings

Figure 5.3 shows the minimum and maximum recorded thermocouple and thermistor readings recorded during the fabrication of the girders. Prior to placing the concrete, the average thermocouple reading had increased to 98°F, approximately 20°F higher than the ambient temperature when the strands were jacked the previous day. This increase in temperature prior to placing the concrete causes a loss in the prestress force prior to casting the concrete. Assuming a 365 ft bed length ($L_o$) with approximately 28 ft of strand not exposed to the increased temperature ($L_1=365-28=337$), and using the coefficient of thermal expansion of the prestressing steel, $\alpha$, of 6.78 με/°F (12.2 με/°C) and an increase in temperature ($\Delta T$) of 20°F, the change in stress in the prestressing steel is given by:
\[ \Delta f = \frac{E \alpha \Delta T L}{L_0} \]  

which indicates a prestressing loss \( \Delta f = 3.6 \text{ ksi} \) may have occurred prior to casting the concrete. This loss is often not considered in design, primarily because the designer has no control over when a girder will be fabricated. If the temperature at the time of casting was lower than at the time the strands were jacked, then a gain in prestressing force would have occurred instead of a loss. This loss is approximately the same magnitude as the loss due to relaxation, which is considered in design. When combined with the relaxation loss, this loss may be significant. This is a type of loss that should be considered and adjusted for in the field at the time of fabrication.

![Figure 5.3 – Temperature Readings during Casting](image)

As shown in Figure 5.3, although the target curing temperature range established by the fabricator was between 145°F and 150°F, the actual temperatures in the bed and in the girders were higher. Two temperature gages were placed through the tarps about mid height of the
specimens and monitored by the fabricators Quality Control person. The maximum temperature recorded by these gages was only 155°F. However, the hottest bed temperature recorded by the thermocouples and the thermistors was 190°F and the hottest concrete temperature recorded was 192°F. Actual temperatures were more than 40°F higher than the fabricator had anticipated. The highest recorded temperatures were in beam VT3, which was near the exit of a steam feed pipe. As expected, since the steam rises under the tarps, higher temperatures were recorded near the top of the specimens.

The actual concrete curing temperatures varied significantly based on location within the prestressing bed, as shown in Appendix G. For the thermocouples, the maximum temperature differential recorded was 46°F, and for the thermistors, the maximum temperature differential recorded was 48°F. Such high temperature differentials during curing may cause the concrete to cure improperly or cause residual strains due to the thermal gradients at the time of detensioning.

5.1.2 Vibrating Wire Gage Readings

The temperature correction for the vibrating wire gages was considered using several different approaches. Figure 5.4 shows the vibrating wire gage readings in one location using three of the approaches. Figure 5.2 shows the location of the vibrating wire gages. A positive reading indicates expansion while a negative reading indicates contraction. Also included is the average thermocouple reading within the girders recorded. Since the gages were unstable during the casting and immediately after due to the external vibration, the first reading was taken approximately one hour after casting was complete, at 2:22 PM. The first series shows the cumulative strain reading corrected for the temperature using a constant value of 4.7 με/°F (8.46 με/°C) for the coefficient of thermal expansion. This is the value determined experimentally using the cylinders cast during fabrication. The second series shows the cumulative strain reading corrected for the temperature using a constant value of 5.78 με/°F (10.4 με/°C) for the coefficient of thermal expansion. This value is recommended by the manufacturer of the vibrating wire gages. Based on the work of Kada, the third series shows the cumulative strain reading corrected for the temperature using a value for the coefficient of thermal expansion of the concrete that varies for the first 12 hours (Kada, et al 2002). It was assumed that the coefficient of thermal expansion would reach an average value of 5.1 με/°F (9.18 με/°C) at an age of 12 hours. Prior to this time, the value would start off as three times this
value at an age of zero and decrease linearly for 12 hours. Theoretically, since there is more free water in the concrete at very early ages, and since water has a much higher coefficient of thermal expansion than concrete, the concrete at early ages will also have a higher coefficient of thermal expansion. The coefficient of thermal expansion for the steel in the vibrating wire gage of 6.78 με/ºF (12.2 με/ ºC) is used for all three cases.

Figure 5.4 – Vibrating Wire Gage 3-3

Since the correction for thermal expansion is made for all gages in the data presented, any changes in strain observed during the curing process should not be due to thermal expansion or contraction. Therefore, the strains observed should most likely be due to shrinkage effects or expansion of the deck due to hydration. It was expected that a slight expansion may occur early due to the hydration of the deck, followed by contraction due to autogenous and drying shrinkage.

All three series indicate that the concrete began to gradually expand after being placed up to about the time the steam was turned on, 4.5 hours. At this time, the two series using the
constant values for the coefficient of thermal expansion indicate that the concrete began to expand more rapidly until approximately 9 hours after casting. However, the series with the variable coefficient of thermal expansion indicates that the concrete rapidly contracted for about 1 ½ hours, and then began to expand. From 9 to 15 hours, the first two series show that the concrete contracted. For the next two hours, the concrete expanded and then began to gradually contract until detensioning.

It is clear that the assumptions used for the coefficient of thermal expansion of the concrete have a significant influence on the cumulative strain readings. At 21 hours after casting, series two indicates approximately zero strain while series one indicates approximately 90 microstrains. The behavior of the series with the variable coefficient of thermal expansion, series three, magnifies the slight decrease seen from 4 to 6 hours in series one and two. After 12 hours, the three series follow a similar trend with only the magnitudes of the strains differing. At detensioning, the change in strain from all three series is approximately the same, since the change in temperature over this short time period was small.

Figure 5.5 shows the average top and bottom center vibrating wire gage readings recorded using an alpha of 4.7 $\mu \varepsilon/\degree F$. Figure 5.2 shows the vibrating wire gage layout. Also included is the average thermocouple reading and the vibrating wire gage reading for the control cylinder. Both the top and bottom readings expand rapidly after the steam is applied at approximately 4.5 hours after casting until they reach a maximum expansion at 8.5 hours after casting. The average top expansion is 138 $\mu \varepsilon$ and the average bottom expansion is 124 $\mu \varepsilon$. The maximum expansion recorded is 224 $\mu \varepsilon$ in gage 1-4; however, this reading appears to drift excessively shortly after casting the concrete. The next highest expansion recorded was 204 $\mu \varepsilon$ in gage 2-4. The average top and bottom readings begin to decrease after the maximum expansion is reached until approximately 16 hours after casting, where an expansion and subsequent contraction occurs. All of the VWG readings are included in Appendix G.

The vibrating wire gage readings for the control cylinder indicate that the cylinder begins to contract after concrete is placed until the steam is applied at 4.5 hours. After this point, the gage follows a similar trend as the other gages. It is possible that autogenous shrinkage may be the primary cause of the early age contraction of the gage in the control cylinder. Autogenous shrinkage occurs due to a volume change caused by the hydration process. It is different than
drying shrinkage which occurs due to a loss of adsorbed water. The potential for autogenous shrinkage is generally considered greater in high performance concretes such as the concrete used for the girders. This occurs because high performance concretes have a lower water to cement ratio. During the hydration process, which occurs rapidly in high performance concrete, the final hydration products produced occupy less space than the materials used to produce them. If there is not enough water present during the hydration, then the reaction will create microstructures that have completed the hydration process even though all of the cementitious materials have not completely hydrated. Pores will be formed within the completed microstructures. If water is not available to fill the pores, then autogenous shrinkage occurs (Bentz, et al 2004).

The cylinder was made according to ASTM specifications; however, it was not exposed to either internal or external vibration as was the concrete used for the girders. This potentially caused the concrete in the cylinder to be less consolidated than the concrete used in the girder. A less consolidated concrete may have a greater potential for autogenous shrinkage because the

Figure 5.5 – Average VWG Readings
concrete will begin to set quicker, reducing the available water to complete the hydration process. Also, the plastic mold used for the control cylinder offers less restraint than the steel forms used for the girders.

From the data, it is impossible to determine the exact time when the concrete reached final set. Most likely, the concrete had set by the time the steam was turned on at 4.5 hours after casting. At this point, heating the system with the steam caused the overall expansion of the girders and the metal formwork which occurred until approximately 8.5 hours after casting. The average temperature in the girders continued to rise after this time, but a contraction occurred despite the rise in temperature. This contraction is most like due to autogenous shrinkage. At approximately 16 hours after casting, the steam was most likely increased, and a subsequent expansion took place due to the thermal expansion of the system. A gradual contraction took place after this point which is most likely due to a combination of autogenous and drying shrinkage.

5.2 Detensioning of Strands

One of the objectives of recording the vibrating wire gages during casting was to be able to estimate the effective prestressing force after detensioning of the strands. By knowing the effective prestressing force, the total prestress losses up to that point can be estimated. The change in the strain measured by the vibrating wire gages immediately before and after detensioning can be used to estimate the change in stress at detensioning. Since the modulus of elasticity is needed for this calculation, a cylinder was tested immediately after the detensioning took place to determine the modulus of elasticity of the girder concrete at that time. The average modulus of elasticity measured at detensioning was 5,200 ksi.

5.2.1 Curvature Measurements

Figure 5.6 shows the bottom center vibrating wire gages immediately before and after detensioning of the strands. Because of the thunderstorm that occurred, the process of detensioning took longer than usual. The points labeled show where the data for the curvature calculations is obtained. The point after detensioning is 80 minutes after the point prior to detensioning. These points were chosen at points where the data had stabilized. As can be seen in the figure, the strain readings were stable prior to and after these points.
Figure 5.6 – Bottom Vibrating Wire Gages at Detensioning

Figure 5.7 – Top Vibrating Wire Gages at Detensioning
Figure 5.7 above shows the top center vibrating wire gages at the same time period as the bottom center vibrating wire gages. These readings are not as stable prior to and after the chosen recording locations. Prior to detensioning, the top gages were contracting and after detensioning, the top gages were expanding. Some of the top strands had been cut at the abutments and left attached between the girders for a short duration because of the thunderstorm. This explains some of the behavior during the 80 minute interval chosen. After detensioning, creep in the concrete due to the prestressing force may be the cause of the expansion in the readings in the top gages.

Table 5.1 shows the predicted effective prestressing stress immediately after detensioning for three methods, the Zia, Scott, Preston, and Workman Method (Z,S,P,W), the PCI Stepwise Method (PCI), and the NCHRP 496 method (Tadros method). The top portion of the table, “As Designed”, shows the values that a designer most likely would predict without modifying the prediction based on the actual field data. The bottom portion of the table, “As Observed”, shows the predicted values based on the field measured modulus of elasticity and the actual amount of time the strands had been stressed prior to detensioning (approximately 4 days instead of the normal 2 days). The as designed jacking stress is based on 0.75 times \( f_{pu} = 270 \) ksi and the as observed jacking stress is based on the actual jacking force of 32 kips. AASHTO equations for relaxation loss are used in the Z,S,P,W method.

<table>
<thead>
<tr>
<th>Method</th>
<th>As Designed</th>
<th></th>
<th></th>
<th>As Observed</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>f Jacking</td>
<td>Relaxation</td>
<td>Elast. Short.</td>
<td>f Release</td>
<td></td>
</tr>
<tr>
<td>Z,S,P,W</td>
<td>202.5</td>
<td>-2.4</td>
<td>-10.8</td>
<td>189.3</td>
<td></td>
</tr>
<tr>
<td>PCI</td>
<td>202.5</td>
<td>-2.1</td>
<td>-11.3</td>
<td>189.1</td>
<td></td>
</tr>
<tr>
<td>NCHRP 496</td>
<td>202.5</td>
<td>-1.8</td>
<td>-11.4</td>
<td>189.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>189.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>f Jacking</td>
<td>Relaxation</td>
<td>Elast. Short.</td>
<td>f Release</td>
<td></td>
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<tr>
<td>Z,S,P,W</td>
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<td>-3.2</td>
<td>-10.1</td>
<td>195.9</td>
<td></td>
</tr>
<tr>
<td>PCI</td>
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<td>-2.9</td>
<td>-12.0</td>
<td>194.3</td>
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<tr>
<td>NCHRP 496</td>
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<td>-2.4</td>
<td>-10.8</td>
<td>196.0</td>
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<td></td>
<td>Average</td>
<td>195.4</td>
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<td></td>
</tr>
</tbody>
</table>
The average as observed effective stress of 195.4 ksi after detensioning was used to calculate the predicted curvature immediately after detensioning. Table 5.2 shows the actual curvature measured at the center of all six girders as well as the predicted curvature.

Table 5.2 – Curvature at Detensioning

<table>
<thead>
<tr>
<th>Girder Mark</th>
<th>Top Gage $\Delta \varepsilon$, $\mu\varepsilon$</th>
<th>Bottom Gage $\Delta \varepsilon$, $\mu\varepsilon$</th>
<th>Measured Curv., $\Phi$ ($10^6$)</th>
<th>Predicted Curv., $\Phi$ ($10^6$)</th>
<th>Percent Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>VT1</td>
<td>-86</td>
<td>-473</td>
<td>-11.7</td>
<td>-10.7</td>
<td>9.7</td>
</tr>
<tr>
<td>VT2</td>
<td>-103</td>
<td>-534</td>
<td>-13.1</td>
<td>-10.7</td>
<td>22.2</td>
</tr>
<tr>
<td>VT3</td>
<td>-106</td>
<td>-499</td>
<td>-11.9</td>
<td>-10.7</td>
<td>11.2</td>
</tr>
<tr>
<td>VT4</td>
<td>-90</td>
<td>-499</td>
<td>-12.4</td>
<td>-10.7</td>
<td>15.8</td>
</tr>
<tr>
<td>VT5</td>
<td>-91</td>
<td>-511</td>
<td>-12.7</td>
<td>-10.7</td>
<td>18.9</td>
</tr>
<tr>
<td>VT6</td>
<td>-82</td>
<td>-522</td>
<td>-13.3</td>
<td>-10.7</td>
<td>24.8</td>
</tr>
<tr>
<td>Average</td>
<td>-93</td>
<td>-507</td>
<td>-12.5</td>
<td>-10.7</td>
<td>17.1</td>
</tr>
</tbody>
</table>

All of the measured curvature values are higher than the predicted curvature value of negative 10.7($10^{-6}$) $\mu\varepsilon$/in. The average measured curvature value is 17.1 percent higher than the predicted value, with a range of 9.7 percent to 24.8 percent higher. A higher negative curvature indicates that there is more compression in the bottom of the girder or more tension in the top of the girder than predicted. More prestressing force below the neutral axis would cause both of these effects. The average measured change in strain at detensioning in the top gage is -93 $\mu\varepsilon$ compression while the predicted change in strain is -15 $\mu\varepsilon$. The average measured change in strain in the bottom gage is -507 $\mu\varepsilon$ compression while the predicted change in strain is -368 $\mu\varepsilon$.

It is unusual to record changes in strain at detensioning that are larger in magnitude in compression than predicted. Many of the studies which have been performed to monitor the stress in the strands during detensioning conclude that the stress in the strand is less than the current methods predict. Less stress in the strands would cause less compression at detensioning. A recent report for the Louisiana Department of Transportation monitored AASHTO Type III girders (very similar to the shape of the PCBT girders used in this study) during fabrication with load cells placed on select strands between the bulkhead and the chucking devices (Roller, et al 2003). The load cells indicated that the stress in the strands decreases more than predicted during the steam curing process. At detensioning, the average force in the strands decreased from an initial jacking force of 30.98 kips to 28.70 kips, a 7 percent loss in force. The authors
concluded that the thermal expansion of the girders during curing caused the loss in the prestressing force.

It is certainly possible that the thermal expansion of the girders could cause the strand outside of the girders to decrease if the detensioning occurs while the girders are in an expanded state. As shown in Figure 5.5, if detensioning had occurred before 28 to 30 hours after casting the concrete, it would have been likely that the expansion of the concrete would have caused a reduced stress in the strands outside of the girders. However, a reduction in stress in the strands outside of the girder does not necessarily indicate a reduction in the stress in the strands within the girder. If the concrete sets early and acts composite with the strands, then the force at detensioning will have already been locked into the girder, and the change in force in the strand outside the girder will not necessarily affect the final force applied to the concrete at detensioning by the prestressing strands.

There are several possibilities that exist to explain why more compression was measured at detensioning than predicted. It is possible that the modulus of elasticity of the actual girder concrete was lower than the measured modulus of the control cylinder. This is unlikely because the control cylinder was measured at the time of detensioning and the modulus of the actual girder concrete is usually higher instead of lower than the modulus of test cylinders.

A second possible explanation is the presence of thermal gradients at the time the concrete set or at the time of detensioning. A study which monitored the temperature profile of Washington State W74MG girders during fabrication recently reported that a temperature gradient that exists when the concrete sets will induce mechanical strains that result in a loss of prestressing force after cooling occurs (Barr, et al 2004). Since a loss in prestressing force is opposite of what is observed, this may not explain the behavior of these girders. Also, it is believed that the concrete set very early, before any significant thermal gradient due to the heat of hydration or steam curing could be established. A significant thermal gradient did exist at the time of detensioning. Figure 5.8 shows the measured thermal gradient in girder VT1 at the time of detensioning. The first diagram shows the total gradient in degrees Fahrenheit and the second shows the differential temperature gradient, derived by subtracting the minimum temperature from all values of the total gradient. Diagram A shows the differential restrained thermal stresses that theoretically occur assuming the modulus of elasticity of the concrete equal to 5,200
ksi and a coefficient of thermal expansion of the concrete of 5.1 με/°F (9.18 με/°C) if fully restrained.

Figure 5.9 shows the final strain profile (in με) that would occur due to the temperature gradient. The Self Equilibrating Stresses (D), in psi, are determined by adding the restrained thermal stresses (A) to the opposite of the restrained axial stresses (-B) and restrained flexural stresses (-C). The free strains (E) are determined by multiplying the temperature gradient by the coefficient of thermal expansion. The Self Equilibrating strains (F) are determined by dividing the Self Equilibrating stresses (D) by the modulus of elasticity. The total strain (G), which must be linear, is the free strain (E) plus the Self Equilibrating Strains (F).

The thermal gradient produces predicted tensile strains of 132 με at the location of the bottom gage and 64 με at the location of the top gage. For girder VT3, a similar thermal gradient, although slightly less in magnitude, existed at the time of detensioning which produces predicted values of 99 με at the location of the bottom gage and 44 με at the location of the top gage. The average predicted top strain is 54 με and the average predicted bottom strain is 116 με. The role of the strains produced by the thermal gradient at detensioning is unclear. If you assume that the strands and the bottom of the prestressing bed were able to restrain the thermal gradient until detensioning, then you would expect the girder to expand when the strands are cut. This would cause positive strains. If the positive strains due to the thermal gradient are added to the negative strains predicted to occur due to detensioning, then the total predicted change in strain would be 39 με for the top gage and -252 με for the bottom gage. These values are even further from the actual values observed than the original predicted values. However, if
you assume that the active force of the strands on the section at detensioning is able to overcome the thermal strains which are already induced in the section, in essence pushing the already expanded section back into compression, then you would subtract the predicted thermal strains from the predicted negative strains due to detensioning. The resulting strains are -69 με for the top gage and -484 με for the bottom gage. These values are only 35 percent and 5 percent lower than the actually observed readings for the top and bottom gages respectively. Even though this second assumption makes the observed data fit the predicted, there is no indication that the active force of the prestressing at detensioning causes greater strains than the strains due to the thermal

Figure 5.9 – Total Strains at Detensioning
The most likely reason that the average strain readings are higher in compression than predicted is possible contraction of the other beams in the pour which were cast the day after the six girder specimens were cast. Three girders each approximately 77 ft long were cast the day after the original 6 test specimens because the clips needed for the fall protection were not available at the time the test specimens were cast. After the forms were removed from these three specimens, moderate vertical cracks were noted near the midspan of each of the three girders. After detensioning, the cracks closed and were less noticeable. The presence of the cracks prior to detensioning indicates that the shrinkage may have occurred during the fabrication. If shrinkage occurred during the curing process, the shrinkage of the girders may have pulled the strands into tension and caused the six test girders to go into tension prior to detensioning. If this occurred, then the amount that the test girders were in tension would show as an additional compressive strain in the vibrating wire gages at detensioning.
6 Static Phase Results and Discussions

6.1 Overview of Static Phase

The purpose of the Static Phase was to monitor the early age development of restraint moments due to curing of the deck, thermal changes, shrinkage, and creep. As described in Section 3.4.2, the outside reactions, Reactions R1 and R4, were monitored along with the electrical resistance gages in the deck and the vibrating wire gages from the time the decks were cast until the service testing began. The restraint moment at the center of the two span system was estimated by multiplying the change in reaction by the span length from centerline of bearing to centerline of bearing (14 ft). See Figure 3.6 for the locations of the strain gages.

Significant restraint moments were observed within the first week of monitoring for the two tests which had a continuity diaphragm. Test 1, which had the extended strands produced higher restraint moments than Test 2, which had the extend 180 degree bent bars. Even though Test 3 did not have a continuity diaphragm, some changes in reactions occurred after casting the continuous deck.

6.2 Development of Early Age Restraint Moments

6.2.1 Test 1 and Test 2

Figure 6.1 shows the changes in reactions and strains for Test 1. Lines are used to connect the data points. Compression is negative for both the reactions and the strains. The reaction values are absolute reactions in kips. The strain gage values are the change in strain from the first reading taken immediately after the deck was cast. At day zero, the deck is cast and the average outside reactions increase from approximately 5.69 kips to 8.96 kips due to the weight of the deck. Almost immediately, the outside reactions begin to increase until a maximum outside reaction of 12.70 kips is recorded at 0.9 days after casting. The reactions then quickly decrease to a value of 7.13 kips at an age of 3.8 days. Over the next 24 days, the reactions slowly decrease to a final value of 5.17 kips at 28 days, when setup for the service testing began. Thermal effects caused by daily fluctuations in temperature cause the reactions to cycle up and down over this time.

The strain gages indicate that the strain increases for the first day, indicating that the gages are being pulled in tension, and then decrease for the remaining time, indicating that the
Figure 6.1 – Changes in Reactions and Strains for Test 1
gages are being compressed. The gages in the top center of the two span unit, gages E5 and E6, decrease the most, approaching 340 με compression at 28 days. The gages in the bottom of the continuity diaphragm, gages E11 and E12, also show significant compression, approaching 290 με compression at 28 days. Gage E1 was most likely damaged during casting the deck and did not function. Also, a wire became loose during the early readings, causing some noise in the electrical resistance gage readings for the early readings of gages E2 through E10.

Figure 6.2 shows the changes in reactions and strains for Test 2. The reactions follow a trend similar to Test 1. Immediately after casting the deck at age zero, the average outside reactions increase from 6.67 kips to 8.55 kips due to the weight of the deck. The reactions begin to increase rapidly until a maximum reaction of 10.35 kips occurs at 0.7 days after casting. The reactions then begin to decrease rapidly until an average reaction of 6.88 kips is reached at 4.2 days. Unlike Test 1, the reactions in Test 2 do not change much from day 4.2 until set up for the service testing began at day 20.

The strains in the electrical resistance gages do not change as much in Test 2 as they do in Test 1. This was unexpected, since the girders for Test 2 were older than the girders for Test 1 at the time the decks were cast. More differential shrinkage should have taken place in Test 2 than in Test 1. Gages E5 and E6, the top center gages, approach a compression of 220 με while the bottom gages in the diaphragm, gages E11 and E12, only approach a compression of approximately 70 με.

The restraint moment derived by multiplying the change in reaction by the distance of 14 ft for Test 1 is twice as much as the restraint moment for Test 2, 52.4 kip-ft versus 25.2 kip-ft, at the early age of 0.9 and 0.7 days, respectively. The restraint moments at the intermediate ages of 3.8 and 4.2 days are similar in magnitude, -25.7 kip-ft and -23.4 kip-ft. At the final age of 28 and 20 days, respectively, Test 1 has nearly twice the restraint moment of Test 2, -53.1 kip-ft and -22.5 kip-ft.

6.2.2 Test 3

Figure 6.3 shows the changes in reactions and strains for Test 3. As expected, since there is no continuity diaphragm, the changes in reaction are much smaller than Tests 1 and 2. After casting the deck, the reactions remain approximately 8.46 kips from the time of casting to 0.9 days after casting. A slight decrease in the reactions then occurs to 7.77 kips at 4.0 days.
Figure 6.2 – Changes in Reactions and Strains for Test 2
Figure 6.3 – Changes in Reactions and Strains for Test 3
The average reactions then remain fairly constant; decreasing only to approximately 7.24 kips at 14 days. Also, the strains do not change as much as Test 1 and 2. The center two gages, gages E7 and E8, approach a value of approximately 90 $\mu$ε compression at 14 days.

6.2.3 Summary of Restraint Moments

Table 6.1 shows a summary of the changes in reactions and resulting moments for each of the three tests. The Net Reaction is the actual reaction recorded. The Delta Reaction is the change in the reaction from one time to the next. The Net Moment is the actual calculated restraint moment using a moment arm of 14 ft. The Delta Moment is the change in the moment from one time to the next.

Some interesting conclusions can be derived from an overall review of the development of the restraint moments in the two tests with the continuity diaphragms. First, it is apparent that some restraint moments develop early, within hours after casting the deck. Many current design procedures do not consider any development of restraint moments until the deck has cured and shrinkage is believed to begin, often at 7 days. The increase in reactions over the first day is consistent with expansion of the deck due to thermal effects. As the deck wants to expand, the composite section wants to rise in the center. However, since the section acts at least partly indeterminate, positive restraint moments are developed in the center and the outside reactions increase.

After approximately 1 day, the outside reactions begin to decrease rapidly. A decrease in reaction indicates that the top of the composite section wants to contract. As the composite section attempts to bend in a positive curvature, the partly indeterminate action causes the reactions at the outside to decrease. For both tests, the outside reactions actually decreased to values less than the reactions when the decks were first cast. This indicates that some negative restraint moment is present even at the early ages of approximately four days.

After four days until the end of monitoring, the reactions did not change as much as expected. This phenomenon has been noticed by other engineers that have stated that once the deck is cast, not much significant bending, or change in curvature, occurs in the section. Since bending is required to cause the development of restraint moments, restraint moments will not develop if bending is not present.
Table 6.1 – Summary of Restraint Moments

<table>
<thead>
<tr>
<th>Test Number 1 Average Outside Reactions and Restraint Moments</th>
<th>Time</th>
<th>Net Reaction</th>
<th>Delta Reaction</th>
<th>Net Moment</th>
<th>Delta Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>kips</td>
<td>kips</td>
<td>kip-ft</td>
<td>kip-ft</td>
</tr>
<tr>
<td>After Casting Deck</td>
<td>-8.96</td>
<td>-3.74</td>
<td>0.00</td>
<td>52.36</td>
<td></td>
</tr>
<tr>
<td>At 0.9 Days</td>
<td>-12.70</td>
<td>5.58</td>
<td>-78.05</td>
<td></td>
<td></td>
</tr>
<tr>
<td>At 3.8 Days</td>
<td>-7.13</td>
<td>1.96</td>
<td>-25.69</td>
<td></td>
<td></td>
</tr>
<tr>
<td>At 28 Days</td>
<td>-5.17</td>
<td></td>
<td>-27.37</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test Number 2 Average Outside Reactions and Restraint Moments</th>
<th>Time</th>
<th>Net Reaction</th>
<th>Delta Reaction</th>
<th>Net Moment</th>
<th>Delta Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>kips</td>
<td>kips</td>
<td>kip-ft</td>
<td>kip-ft</td>
</tr>
<tr>
<td>After Casting Deck</td>
<td>-8.55</td>
<td>-1.80</td>
<td>0.00</td>
<td>25.20</td>
<td></td>
</tr>
<tr>
<td>At 0.7 Days</td>
<td>-10.35</td>
<td>3.47</td>
<td>-48.58</td>
<td></td>
<td></td>
</tr>
<tr>
<td>At 4.2 Days</td>
<td>-6.88</td>
<td>-0.06</td>
<td>-23.38</td>
<td></td>
<td></td>
</tr>
<tr>
<td>At 20 Days</td>
<td>-6.94</td>
<td></td>
<td>-22.54</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test Number 3 Average Outside Reactions and Restraint Moments</th>
<th>Time</th>
<th>Net Reaction</th>
<th>Delta Reaction</th>
<th>Net Moment</th>
<th>Delta Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>kips</td>
<td>kips</td>
<td>kip-ft</td>
<td>kip-ft</td>
</tr>
<tr>
<td>After Casting Deck</td>
<td>-8.45</td>
<td>-0.02</td>
<td>0.00</td>
<td>0.28</td>
<td></td>
</tr>
<tr>
<td>At 0.9 Days</td>
<td>-8.47</td>
<td>0.70</td>
<td>0.28</td>
<td></td>
<td></td>
</tr>
<tr>
<td>At 4.0 Days</td>
<td>-7.77</td>
<td>-9.52</td>
<td>28.53</td>
<td></td>
<td></td>
</tr>
<tr>
<td>At 14 Days</td>
<td>-7.24</td>
<td>-16.87</td>
<td>-7.35</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

6.2.4 Observed Curvatures

Since restraint moments can not develop in a continuous system unless a member in the system bends, or undergoes a change in curvature, the changes in curvature from the time the decks were casts were monitored for the three tests. Figure 6.4 shows a plot of the changes in
curvature, in με/in. units, for each of the three tests at the same time the maximum restraint moments occur. A positive curvature indicates positive bending. At the center of the girders and over the support near the diaphragm, the change in curvature is determined from the vibrating wire gage readings. At the center of the continuity diaphragms, the change in curvature is determined from the electrical resistance gages.

The results of Test 1 and Test 2 are similar. At the early age when thermal effects cause the reactions to increase, negative curvature values are recorded in the center of the girders and positive curvature values are recorded at the center of the continuity diaphragm. As the thermal effects decrease and shrinkage begins to dominate, the curvatures in the center of the girders goes from negative to positive for both tests. This makes sense since the sum of the curvature caused by the shrinkage moment and the restraint moment should be negative. At the diaphragm for both tests, the curvature measurements remain positive, which does not match the predicted curvature for this type of system. This will be

Figure 6.4 – Curvature Diagrams
discussed more in the following sections. For Test 3, the measured curvatures are negative for all stages. The negative values near the center of the two span system make sense if some restraint is established, which was noticed in the changes in reactions. However, the negative curvatures at the center of the two span setup do not match conventional analyses.

6.3 Conventional Prediction of Early Age Restraint Moments

For precast, prestressed multi-girder construction made continuous with a continuity diaphragm, most designers typically only consider restraint moments due to creep and shrinkage of the concrete. Many of the design procedures also consider shrinkage to start at 7 days. Since this study as well as the recent NCHRP 519 study has noticed the development of restraint moments in the continuous system as early as one day after the casting of the deck, it is apparent that the typical design procedure which waits to consider shrinkage after an age of 7 days needs reconsidering (Miller, et al 2004).

Since positive restraint moments develop at an age of approximately one day, expansion of the deck due to thermal effects is most likely occurring. After that time, shrinkage of the deck dominates causing the development of negative restraint moments. A conventional analysis of the actual system is performed to predict the restraint moments due to the superposition of the thermal effects and the shrinkage effects. It is assumed that the deck and the girder act compositely from the time the deck is cast. Since the observed reactions begin to change almost immediately, some composite action is occurring early on. Actual values for the properties of the deck and girder concrete are used in the analysis. Both the thermal effects and the shrinkage effects cause changes in curvature in the system which are resisted by the restraint moments. Therefore, two restraint moments are determined and added together for the final predicted restraint moment. A table comparing the predicted and the observed strains and restraint moments is included in Section 6.3.3. The moduli of elasticity for the deck and concrete used for the analysis are shown in Table 6.2.

6.3.1 Thermal Restraint

The procedure outlined in the Section 2.6.2.3 Thermal Effects is used to determine the final distribution of stresses and strains on the composite section due to a nonlinear temperature gradient. Figure 6.5 shows an example calculation for Test 1 at 0.9 days. The temperature
Table 6.2 – Moduli of Elasticity used in Analyses

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Age of Concrete Days</th>
<th>Deck Modulus E, ksi</th>
<th>Girder Modulus E, ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test #1</td>
<td>0.9</td>
<td>3620</td>
<td>5730</td>
</tr>
<tr>
<td></td>
<td>3.8</td>
<td>3810</td>
<td>5760</td>
</tr>
<tr>
<td></td>
<td>28</td>
<td>5070</td>
<td>5710</td>
</tr>
<tr>
<td>Test #2</td>
<td>0.7</td>
<td>2470</td>
<td>5570</td>
</tr>
<tr>
<td></td>
<td>4.2</td>
<td>2690</td>
<td>5570</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>4480</td>
<td>5590</td>
</tr>
</tbody>
</table>

profile shown in the example is the actual temperature profile recorded at 0.9 days with the minimum temperature subtracted from all values. Since the temperature profile indicates that the composite section heats up, the first step is to assume that the section is fully restrained. The product of the modulus of elasticity, coefficient of thermal expansion, temperature, and section width are integrated over the depth of the section to get the restraining force, \(P_{\text{rescomp}} = 265.8 \text{ kips}\). The restraining moment, \(M_{\text{rescomp}} = 282.5 \text{ kip-ft}\), is obtained by integrating the modulus of elasticity, the coefficient of thermal expansion, temperature, section width, and the distance from the centroid of the composite section over the depth of the composite section. This moment can be checked by determining an approximate moment by assuming that only the deck expands and multiplying the increase in temperature of the deck (18.5°F) by the coefficient of thermal expansion (5.1E-6 in/in/ºF), the area of the deck and bolster (610.5 in.²), the modulus of the elasticity of the deck (3620 ksi), and the distance to the composite centroid (18.14 in.). This produces a moment equal to 315 kft, which is approximately 12 percent higher than the actual moment.

Once the restraining force and moment are determined, the stresses, strains, and curvature due to the thermal gradient can be determined. The example shows how the restrained thermal, axial, and flexural stresses are determined for the girder. The self equilibrating stresses are then determined by adding together the restrained thermal stresses and the opposite of the restrained axial and flexural stresses. The final strain distribution is then determined by adding the strains caused by free thermal expansion to the self equilibrating strains. Since the thermal gradient is
Given the Following Properties for Test 1 at an age of 0.9 Day:

\[
\begin{align*}
\text{Ed} &= 3620 & \text{Eg} &= 5730 & n &= \frac{\text{Ed}}{\text{Eg}} & n &= 0.632 & \alpha &= 0.0000051 & \text{ad} &= \alpha & \text{Acomp} &= 1133 & \text{Icomp} &= 402528 & \text{yd} &= 9.362
\end{align*}
\]

\text{yd} \text{ is the distance from the noncomposite to composite cg}

\[
\text{Prescomp} = \left\{ \begin{array}{l}
\int_{-21.23-\text{yd}}^{15.23-\text{yd}} \text{Eg} \cdot \alpha \cdot T1 \cdot b1 \ dy + \\
\int_{-22.23-\text{yd}}^{15.23-\text{yd}} \text{Eg} \cdot \alpha \cdot T2(y) \cdot b1 \ dy + \\
\int_{-21.23-\text{yd}}^{15.23-\text{yd}} \text{Eg} \cdot \alpha \cdot T3(y) \cdot b2(y) \ dy \\
\int_{-8.73-\text{yd}}^{15.27-\text{yd}} \text{Eg} \cdot \alpha \cdot T3(y) \cdot b3(y) \ dy + \\
\int_{-12.23-\text{yd}}^{18.77-\text{yd}} \text{Eg} \cdot \alpha \cdot T3(y) \cdot b4(y) \ dy + \\
\int_{-21.77-\text{yd}}^{15.27-\text{yd}} \text{Eg} \cdot \alpha \cdot T4(y) \cdot b5(y) \ dy + \\
\int_{-17.27-\text{yd}}^{18.77-\text{yd}} \text{Eg} \cdot \alpha \cdot T4(y) \cdot b6(y) \ dy + \\
\int_{-21.77-\text{yd}}^{15.27-\text{yd}} \text{Eg} \cdot \alpha \cdot T5(y) \cdot b7(y) \ dy \\
\int_{22.77-\text{yd}}^{24.27-\text{yd}} \text{Eg} \cdot \alpha \cdot T6(y) \cdot b7 \ dy + \\
\int_{21.77-\text{yd}}^{22.77-\text{yd}} \text{Eg} \cdot \alpha \cdot T7(y) \cdot b8 \ dy + \\
\int_{24.27-\text{yd}}^{31.77-\text{yd}} \text{Eg} \cdot \alpha \cdot T7(y) \cdot b9 \ dy
\end{array} \right\}
\]

\text{Prescomp} = 265.8

\[
\text{Mrescomp} = \left\{ \begin{array}{l}
\int_{-21.23-\text{yd}}^{15.23-\text{yd}} \text{Eg} \cdot \alpha \cdot T1(y) \cdot b1 \ dy + \\
\int_{-22.23-\text{yd}}^{15.23-\text{yd}} \text{Eg} \cdot \alpha \cdot T2(y) \cdot b1 \ dy + \\
\int_{-21.23-\text{yd}}^{15.23-\text{yd}} \text{Eg} \cdot \alpha \cdot T3(y) \cdot b2(y) \ dy + \\
\int_{-8.73-\text{yd}}^{15.27-\text{yd}} \text{Eg} \cdot \alpha \cdot T3(y) \cdot b3(y) \ dy + \\
\int_{-12.23-\text{yd}}^{18.77-\text{yd}} \text{Eg} \cdot \alpha \cdot T3(y) \cdot b4(y) \ dy + \\
\int_{-21.77-\text{yd}}^{15.27-\text{yd}} \text{Eg} \cdot \alpha \cdot T4(y) \cdot b5(y) \ dy + \\
\int_{-17.27-\text{yd}}^{18.77-\text{yd}} \text{Eg} \cdot \alpha \cdot T4(y) \cdot b6(y) \ dy + \\
\int_{-21.77-\text{yd}}^{15.27-\text{yd}} \text{Eg} \cdot \alpha \cdot T5(y) \cdot b7(y) \ dy \\
\int_{22.77-\text{yd}}^{24.27-\text{yd}} \text{Eg} \cdot \alpha \cdot T6(y) \cdot b7 \ dy + \\
\int_{21.77-\text{yd}}^{22.77-\text{yd}} \text{Eg} \cdot \alpha \cdot T7(y) \cdot b8 \ dy + \\
\int_{24.27-\text{yd}}^{31.77-\text{yd}} \text{Eg} \cdot \alpha \cdot T7(y) \cdot b9 \ dy
\end{array} \right\}
\]

\text{Mrescomp} \quad \text{yd} \quad \text{yd} \quad \text{yd} 

\frac{12}{\text{Mrescomp}} = 282.49

(Continued on Next Sheet)
Thermally Induced Stresses and Strains on Girder

**Restrained Thermal Stresses**
\[ \sigma_{ttop} = -E_g \cdot \alpha \cdot T_6 \quad \sigma_{tbot} = -E_g \cdot \alpha \cdot T_1 \quad \sigma_{tbot} = 0 \]

**Restrained Axial Stresses**
\[ \sigma_{pres} = \frac{-\text{Prescomp}}{\text{Acomp}} \quad \sigma_{pres} = -0.235 \]

**Restrained Flexural Stresses**
\[ \sigma_{mrestop} = \frac{-\text{Mrescomp} \cdot (45 - ycb)}{I_{comp}} \quad \sigma_{mrestop} = -0.113 \]
\[ \sigma_{mresbot} = \frac{\text{Mrescomp} \cdot ycb}{I_{comp}} \quad \sigma_{mresbot} = 0.266 \]

**Self Equilibrating Stresses**
\[ \text{SEtop} = \sigma_{ttop} - \sigma_{pres} - \sigma_{mrestop} \quad \text{SEtop} = -0.135 \]
\[ \text{SEbot} = \sigma_{tbot} - \sigma_{pres} - \sigma_{mresbot} \quad \text{SEbot} = -0.031 \]

**Thermally Induced Free Strain**
\[ \varepsilon_{freetop} = \alpha \cdot T_6 \quad \varepsilon_{freetop} = 0.000084 \]
\[ \varepsilon_{freebot} = \alpha \cdot T_1 \quad \varepsilon_{freebot} = 0 \]

**Final Strain Distribution**
\[ \varepsilon_{finaltop} = \varepsilon_{freetop} + \frac{\text{SEtop}}{E_g} \quad \varepsilon_{finaltop} = 0.000061 \]
\[ \varepsilon_{finalbot} = \varepsilon_{freebot} + \frac{\text{SEbot}}{E_g} \quad \varepsilon_{finalbot} = -0.000005 \]

**Final Curvature**
\[ \Phi_c = -\left( \frac{\varepsilon_{finaltop} - \varepsilon_{finalbot}}{45} \right) \quad \Phi_c = -1.4697 \times 10^{-6} \]
\[ M_{check} = -\frac{\Phi \cdot E_g \cdot I_{comp}}{12} \quad M_{check} = 282.49 \]

Figure 6.5 – Thermal Gradient Example
nonlinear, and plane sections must remain plane, free expansion can not occur due to the gradient. It is best to think of the self equilibrating strains as the strains that are added to the free expansion strains so that the section will have a final linear strain profile. The final curvature is then determined from the difference of the top and bottom strains divided by the section depth. Similar calculations can be done for the deck, and the final curvature of the deck will equal the final curvature of the girder.

Once the equivalent moment, or curvature times the modulus of elasticity and moment of inertia, is determined, the equivalent moment is applied to the continuous system to determine the restraint moment. For the two span system tested, the Conjugate-Beam Method is an efficient method for determining the restraint moment. The method recognizes the fact that the shear in a conjugate beam loaded with the M/EI of the real beam provides the slope of the real beam. This means that the reaction of the conjugate beam loaded with the M/EI is the end slope or rotation for the real beam. As shown in Figure 6.6, the conjugate beam for a real simple span beam is also a simple span beam. In part “A” the equivalent moment is divided by the modulus of elasticity of the girder and the moment of inertia of the composite section and applied to the conjugate beam. The end reaction R, which is also the end rotation, is equal to 6.0454M/EgIc. In part “B” a restraint moment Mres is then applied at the same location and the end reaction is determined. Since for a two span continuous system, the center rotation should be zero, the two reactions must equal each other. Setting the reactions equal to each other and solving for the restraint moment provides a restraint moment \( M_{res} = 349.5 \text{ kft} \).

The final strains can be determined by adding the final strain distribution due to the thermal gradient to the strains due to the restraint moment.
6.3.2 Differential Shrinkage

As concrete cures, it is difficult to separate the expansion due to thermal effects and the early age shrinkage. However, from the shrinkage specimens monitored during the static phase and the electrical resistance gages in the decks, there is evidence that significant shrinkage occurs early. For Tests 2 and 3 shrinkage specimens, more shrinkage occurs than predicted by ACI 209. For Test 1 shrinkage specimens, which were monitored starting at 7 days, the shrinkage observed is similar to the ACI predicted shrinkage starting at 7 days. For Tests 1 and 2, the electrical resistance gages indicate more shrinkage than conventional analyses predicts. Therefore, some shrinkage most likely occurs early. To account for the early age shrinkage, the ACI model for predicting shrinkage is modified to assume that shrinkage begins at age zero instead of age 7. This predicts that approximately 15 με shrinkage occurs within the first day. When added to the thermal expansion that occurs in the first day, the net movement in the deck is still expansion.

Figure 6.7 shows a differential shrinkage example for the Composite Section Method, one of the three methods which can be used to determine the final stresses, strains, and curvature of a composite section due to a differential shrinkage of the deck. All three methods assume that the curvature of the deck and girder will be the same. The Composite Section Method and the Separate Section Method use different approaches but arrive at the same results. These methods can be used on a composite section that undergoes differential shrinkage only. The third method, called the Trost’s Method at Virginia Tech, can consider creep, shrinkage, and aging effects of the concrete in addition to moments and forces due to applied loads and prestressing forces and losses. This method requires the solution of a set of eight simultaneous linear equations.

The Composite Section method is solved for a differential shrinkage of 15 με occurring on Test 1 at 0.9 days. Once composite properties are determined, stresses and strains in the deck, section 1, and the girder, section 2, can be determined directly. The terms for the method are in Section 2.6.2.4. The method assumes that at the interface of the deck and girder that the driving differential shrinkage will cause the girder to go into compression and the deck to go into tension. This can be checked by adding the absolute value of the shrinkage in the bottom of the deck (6 με) to the absolute value of the shrinkage at the top of the girder (9 με) to get the total
Given the Following for the Composite Section Method

<table>
<thead>
<tr>
<th>Deck Properties</th>
<th>Girder Properties</th>
<th>Composite Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_1 = 610.5$</td>
<td>$I_1 = 3807$</td>
<td>$E_1 = 3620$</td>
</tr>
<tr>
<td>$A_2 = 747.3$</td>
<td>$I_2 = 207554.087$</td>
<td>$E_2 = 5730$</td>
</tr>
<tr>
<td>$Ac = 1132.9$</td>
<td>$Ic = 402346$</td>
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</tr>
</tbody>
</table>

For Shrinkage of 15 microstrains:

$\Delta = 0.000015$  
$Q = \Delta A_1 E_1$  
$Q = 33.15$  
$yc = 18.138$  
$yc1 = 13.408$  
$yc = 22.408$  
$ycb = 31.592$

Stresses and Strains in the Deck

$\sigma_{1t} = \frac{Q}{A_1} + \left( \frac{-Q \cdot yc \cdot yc1}{Ic} \right) \frac{E_1}{E_2}$  
$\sigma_{1t} = 0.015$  
$\sigma_{1b} = \frac{Q}{A_1} + \left( \frac{-Q \cdot yc \cdot yc1}{Ic} \right) \frac{E_1}{E_2}$  
$\sigma_{1b} = 0.023$

$\epsilon_{1t} = \frac{\sigma_{1t}}{E_1}$  
$\epsilon_{1t} = 0.000004$  
$\epsilon_{1b} = \frac{\sigma_{1b}}{E_1}$  
$\epsilon_{1b} = 0.000006$

Stresses and Strains in the Girder

$\sigma_{2t} = \frac{-Q \cdot yc \cdot yc1}{Ic}$  
$\sigma_{2t} = -0.049$  
$\sigma_{2b} = \frac{-Q \cdot yc \cdot yc1}{Ic}$  
$\sigma_{2b} = 0.018$

$\epsilon_{2t} = \frac{\sigma_{2t}}{E_2}$  
$\epsilon_{2t} = -0.000009$  
$\epsilon_{2b} = \frac{\sigma_{2b}}{E_2}$  
$\epsilon_{2b} = 0.000003$

Check that Strain in the Bottom of Deck plus Strain in Top of Girder sum to the Applied Shrinkage of 15 microstrain

$\epsilon_{1b} - \epsilon_{2t} = 0.000015$  
OK, Equals 15 microstrain

Determine Curvature and Moment

$\Phi = \frac{\epsilon_{2b} - \epsilon_{2t}}{45}$  
$\Phi = 2.608 \times 10^{-7}$  
$M = Q \cdot yc$  
$M = 50.11$

Check Curvature and Moment with Each Other

$Mecheck = \frac{\Phi \cdot E_2 \cdot Ic}{12}$  
$Mecheck = 50.11$  
OK

Figure 6.7 – Differential Shrinkage Example
differential shrinkage (15 με). Once the strains are determined, the difference in the strains at the top and bottom of a section divided by the height of the section is the curvature. The curvature of the deck and the girder must be the same. The curvature or the equivalent moment of \( M=50.1 \text{ kft} \) is then used to determine the restraint moment. The restraint moment is calculated the same way the thermal restraint moment using the Conjugate Beam method. A restraint moment of -62.0 kft is determined for this example.

Since the change in reaction is only influenced by restraint moments, adding the restraint moment due to the thermal effects (349.5 kft) and the restraint moment due to the shrinkage effects (-62.0 kft) gives a total restraint moment of 287.5 kft. The actual restraint moment observed is 52.4 kft, indicating that the method overestimates the restraint moment by a factor of 5.49.

6.3.3 Comparison of Observed versus Predicted Restraint Moments

Using the conventional analyzes shown in the previous two sections, the restraint strains at the center of the two span section and the restraint moments are predicted for Test 1 and Test 2. Table 6.3 shows a summary of the predicted strains and moments and a ratio of the predicted divided by the observed values. The actual thermal gradients observed were used for all cases. The ACI Predicted shrinkage values of 15 με, 55 με, 190 με, and 232 με were used for 1, 4, 20, and 28 days respectively. For most cases, the predicted strains are less than the observed strains at the center of the two span system. However, the predicted restraint moments are much more

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Age</th>
<th>Gage Location</th>
<th>A Strain (με) Predicted</th>
<th>B Strain (με) Observed</th>
<th>A/B Ratio</th>
<th>C Moment (kft) Predicted</th>
<th>D Moment (kft) Observed</th>
<th>C/D Ratio</th>
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</thead>
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</table>
than observed, especially at 20 and 28 days, where the predicted restraint moments are 21 to 39 times more than the observed restraint moments. Since the predicted values and observed values differ so much, a modified approach needs to be taken to predict behavior of the continuous system.

6.4 Modified Approach for Prediction of Early Age Restraint Moments

Stress relief occurring at the top of the girder from the shrinkage may allow for the observed restraint moments to be less than the predicted restraint moments. The stress relief can be modeled as a system not acting fully composite or continuous until it reaches a certain age. Partial continuity or composite action may allow for some restraint moments to develop, but not the full restraint moments predicted. Also, after the early age, thermal effects diminish and differential shrinkage should become the primary mechanism causing any restraint moments. Creep effects due to self weight and prestressing force should be minimal over a short duration. The creep effects due to self weight counteract the creep effects due to the prestressing force, so any minimal creep should not cause significant rotation. Since the differential shrinkage should be the driving influence, the amount of differential shrinkage needs to be investigated. The reinforcing in the deck should help to resist the shrinkage occurring in the deck. As the deck begins to shrink, the reinforcing goes into compression, producing restraining forces.

A modified approach is used to predict the restraint moments observed in Tests 1 and 2. The degree of continuity or composite action is modeled at all ages and the amount of differential shrinkage is estimated by considering the shrinkage in the girder that occurs during the testing phase. The reinforcing in the deck is considered, but not within the model. An equivalent thermal strain is added to the predicted differential shrinkage and the quantity multiplied by a factor estimating the amount of composite action present. The final equivalent strain is then applied to the composite section using the Composite Section Method, and the restraining moment determined using the Conjugate Beam Method.

6.4.1 Predicted Composite/Continuous Action

The amount of composite or continuous action at any time t in days after casting the deck is estimated using the ACI equation for Concrete Compressive Strength versus Time in ACI 209 (ACI 2002). The equation is intended to provide the percent of ultimate compressive strength of concrete based on the age of the concrete t in days, and a ratio $a/\beta$, the age of the concrete where
the strength is half of the ultimate compressive strength. The formula is:

$$\text{Ratio} = \frac{t}{a + \frac{t}{\beta}}$$  \hspace{1cm} (6.1)

Table 2.2.1 in ACI 209 recommends a value of $a/\beta$ of 4.71 for moist cured concrete with type I cement. This ratio is determined at all ages and used to estimate the percentage of the full action.

### 6.4.2 Predicted Total Strain Changes

The procedure for predicting the total change in strain in the deck adds the predicted thermal strains to the predicted shrinkage strains and reduces the quantity by the ratio of the predicted composite action. First, the predicted thermal strain at each point in time is estimated by taking the average temperature in the deck and subtracting the minimum temperature in the composite section. The deck temperatures observed are usually within a degree of each other, so taking an average temperature seems appropriate. The change in temperature in the deck is then multiplied by the coefficient of thermal expansion of the deck concrete to produce the change in thermal strain. A coefficient of thermal expansion of $5.1 \text{ με/°F}$ is used for the deck concrete.

The predicted shrinkage of the deck and the girder are determined using the ACI predicted values. The deck shrinkage is assumed to start at day zero, and the actual girder ages are used to determine the girder shrinkage. The amount of girder shrinkage is then subtracted from the deck shrinkage at each point in time to produce the differential shrinkage. The thermal expansion is added to the differential shrinkage and the quantity multiplied by the ratio of composite action.

Figure 6.8 shows the total equivalent thermal strains (top two curves), the ACI predicted deck shrinkage starting at day zero (the bottom curve) and the final modeled strains (the middle two curves) used to predict the restraint moments for Test 1 and Test 2. The temperature profiles for both tests are similar, increasing to an equivalent thermal strain of approximately 100 με expansion at an age of one day and then decreasing to less than 10 με after an age of 7 days. The models predict a net expansion of only 14 με at an age of one day instead of the 100 με of pure thermal expansion. The predicted values from the models at the same ages used in the conventional analysis are determined and used to predict the restraint strains and moments using the Composite Section Method and the Conjugate Beam Method. The modulus of elasticity for
the deck and girder used in the analysis are 3605 ksi and 5098 ksi, the predicted modulus of elasticity using the 28 day strength. The ACI Predicted Shrinkage shown is the same as the predicted shrinkage used for the conventional analysis.

Figure 6.8 – Predicted Strains with Modified Approach

6.4.3 Comparison of Observed Versus Predicted Restraint Moments

Table 6.4 shows a summary of the observed versus predicted restraint strains and moments for the modified approach. The modeled strains of 14 με, -7 με, and -143 με at an age of 0.9, 3.8, and 28 days respectively are used for Test 1 and the modeled strains of 13 με, -12 με, and -137 με at an age of 0.7, 4.2, and 20 days respectively are used for Test 2.

For most cases, the predicted strains are still less than the observed strains. However, the predicted restraint moment and the observed restraint moment for Test 1 at the early ages of 0.9 and 3.8 days are nearly the same, with ratios of 1.05 and 1.07. The predicted moment and the observed moment for Test 2 are not as good, with ratios of 2.03 and 2.02 for the early ages of 0.7 and 4.2 days. Compared to the conventional analysis, this method is much better at predicting early age restraint moments at the early ages when thermal influences dominate. For the later
Table 6.4 – Modified Analysis Summary

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Age</th>
<th>Gage Location</th>
<th>A Strain (με) Predicted</th>
<th>B Strain (με) Observed</th>
<th>A/B Ratio</th>
<th>C Moment (kft) Predicted</th>
<th>D Moment (kft) Observed</th>
<th>C/D Ratio</th>
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</tr>
</tbody>
</table>

ages of 28 and 20 days, the predicted moments are still greater than observed, with ratios of 10.61 and 23.95, but still better than the conventional analysis which produces ratios of 18.68 and 39.29 at the same ages.

Creep of the concrete was not considered in the above two procedures, but can be applied by multiplying the final restraint moments by the reduction due to creep using the Rate of Creep Method. For loads that are slowly applied, such as the creep due to the differential shrinkage, the restraining moments should be reduced by the quantity \((1-e^{-\Phi})/\Phi\). For Test 1, the creep coefficient, \(\Phi\), determined by ACI is 0.707 at 28 days and 0.405 at 7 days. Taking the difference of the creep coefficients, 0.302, and inserting in the Rate of Creep equation gives a reduction factor of 0.86. Using this factor will only decrease the predicted restraint moments at 28 days by 14 percent. This improves the analysis, but not by much. For Test 2, using 20 and 7 days gives a factor of 0.90, a 10 percent decrease. Even if the creep is assumed to occur from 0 to 28 days in Test 1, the factor is 0.72, giving a 28 percent reduction. For this case, the ratio of predicted to observed restraint moment is reduced from 10.61 to 7.64.

By comparing the two analyses, it can be concluded that conventional analysis should not be used to estimate restraint moments at early ages. The modified analysis which accounts for the fact that the section may not be fully composite does a better job at predicting the restraint moments at early ages when thermal effects dominate, which for these tests is less than 7 days.

After the early period where thermal effects dominate, not much restraint moment
develops in the two tests. This has been observed in the field where engineers have stated that once the deck is made composite, that the system does not bend or undergo as much curvature as conventional design procedures predict. Since the observed shrinkage in the electrical resistance gages attached to the bars is higher than predicted, there is evidence that shrinkage of the system is occurring; however, it is not manifesting itself as differential shrinkage which causes bending or changes in curvature.

6.4.4 Reinforcing in the Deck

The reinforcing in the deck acts to reduce the potential shrinkage of the deck by going into compression and applying equal outward forces on the deck. Three methods that can be used to estimate the influence that the reinforcing has on the deck are the Effective Modulus Method, the Rate of Creep Method, and the Trost’s Age Adjusted Rate of Creep Method. All three of these methods were investigated to determine the influence of the reinforcing steel in the deck. The results are similar, so only the Effective Modulus Method is presented for the case Test 1 at 28 days.

The Effective Modulus Method assumes that if a reinforced deck undergoes shrinkage that equal and opposite forces will develop in the concrete and reinforcing steel, with the concrete going into tension and the reinforcing steel going into compression. The final strain in the steel, $\varepsilon_s$, can be determined from the following:

$$\varepsilon_s = \frac{A_c \varepsilon_{shr}}{A_c + A_s n'}$$

where for this example $A_c$ is the net area of the deck and bolster concrete (604.92 in$^2$), $A_s$ is the area of the reinforcing steel (5.58 in$^2$), $\varepsilon_{shr}$ is the shrinkage strain (143 $\mu\varepsilon$), $E_c$ is the modulus of elasticity of the concrete (3605 ksi), $E_s$ is the modulus of elasticity of the steel (29000 ksi), and $\Phi$ is the creep coefficient (0.71). Solving for the strain in the steel indicates that the final strain is $127 \mu\varepsilon$ compression. This means that the actual deck shrinks 127 microstrains instead of 143 microstrains. Using the revised shrinkage value of 127 microstrains and the Composite Section Method predicts a restraint moment of 500.5 kft while the actual observed restraint moment is 53.1 kft. Including the reinforcing steel in the deck improves the prediction, but the ratio of the predicted moment to the observed moment is still high at 9.43 instead of the previous ratio.
without including the reinforcing in the deck at 10.61.

For the 6 ft width deck, 18 No. 5 bars were used in the deck. This makes the ratio of the area of reinforcing in the deck to the total deck and bolster area equal to 0.009, which is relatively low if compared to a column. But, for a deck made continuous over an interior support, the amount of reinforcing steel is appropriate, and is based on the expected ultimate negative live load due to traffic.

6.4.5 Other Possible Explanations of Restraint Moments Observed

The modified approach for predicting restraint moments at early ages when thermal influences are dominate works reasonably well for early ages. However, after an age of approximately 7 days, the method does not predict restraint moments adequately. In spite of the fact that shrinkage is measured in the shrinkage specimens and in the electrical resistance gages in the deck, not much net differential shrinkage appears to occur after approximately 7 days. Other reasons for this phenomenon are proposed.

**Softening at Diaphragm and Girder Interface** - It is possible that a region of reduced stiffness forms at the interface of the end of the girder and the continuity diaphragm. As will be discussed in Section 7.2, the end of girder and diaphragm interface acts as a bond break since the concrete for the diaphragm is placed against the already cured ends of the girders. Cracking at the interface would cause the moment of inertia of the section to decrease at the interface. A decreased moment of inertia would then cause less restraint moment for a given end rotation.

However, no cracking was observed at the interface for either test during the Static Phase. For the creation of positive restraint moment which occurred within the first day, it is possible that cracking at the bottom of the section could have occurred while the formwork was still in place. Any cracks that may have formed could have then closed up because of the negative restraint moment that began to develop following the first day after casting the deck. Any cracking that would reduce the moment of inertia of the section for the negative restraint moment would require cracks to form in the top of the section in the deck. No cracking was noticed in the deck, and the electrical resistance gages which were located near the top of the deck went into compression, indicating that tensile cracks most likely did not form in that location. Therefore, even though softening may have occurred within the first day for the positive restraint moments, it is unlikely to have occurred for the negative restraint moments that developed after
that time. The modified approach which accounts for the effectiveness of the composite action is the best predictor of the restraint moments at the early ages and a model incorporating the possible softening effect is not warranted for the early age prediction.

**Girder and Slab Shrink at the Same Rate** – If the girder and slab were to undergo shrinkage at the same rate, then no differential shrinkage would occur. According to Townsend, the ACI Modified by Huo Method does the best job at predicting the shrinkage of a high strength concrete similar to the concrete used in the test girders (Townsend 2002). This method was used to predict the shrinkage of the deck and the girder for both Test 1 and Test 2. The method does produce slightly better results than the modified approach which uses the ACI predictions for shrinkage. However, cumulative differential strains at 28 days for Test 1 and 20 days for Test 2 are still significant, 133 \(\mu\varepsilon\) and 136 \(\mu\varepsilon\) respectively versus the 143 and 137 used in the modified approach. Predicted restraint moments using the ACI Modified by Huo Method assuming shrinkage starts at day zero are slightly improved at the later ages (28 and 20 days) but are actually worse at the intermediate ages (3.8 and 4.2 days). No significant difference is noticed for the predicted moments at the earliest age (0.9 and 0.7 days).

The correction factor for the volume to surface ratio has a significant influence on the predicted ultimate shrinkage. Since shrinkage is typically considered to start at the end of moist curing, then any autogenous shrinkage that takes place prior to the end of curing is typically not considered in the models. The approach presented assumes that shrinkage starts at day zero to allow for some autogenous shrinkage to be predicted. Since the autogenous shrinkage does not depend on the loss of water, it is not dependent on the relative humidity and should not be influenced by the volume to surface ratio. For the modified approach, the actual volume to surface ratio of the shrinkage specimens of 3.0 was used. For considering the ACI Modified by Huo Method, the actual deck volume to shrinkage ratio of 5.3 was used. Using the ratio of 3.0 in the ACI Modified by Huo Method increases the ultimate shrinkage of the deck, producing even more differential shrinkage.

**Deck Shrinks more at the Top than at the Bottom** – Branson describes several different methods in his book which can be used to determine the ultimate change in curvature of a singly reinforced beam subject to shrinkage (Branson 1977). One of the methods, the Miller, et al Empirical Method, assumes that the concrete on the top of a singly reinforced beam will
shrink the same amount as the free shrinkage of the concrete. The curvature of the beam, $\Phi_{sh}$, is then determined as a function of the free shrinkage, $\varepsilon_{sh}$, the strain in the steel due to the shrinkage, $\varepsilon_s$, and the distance from the top of the deck to the centroid of the steel, $d$, by the following:

$$\Phi_{sh} = \frac{\varepsilon_{sh} - \varepsilon_s}{d} \quad (6.3)$$

For the ratio of the strain in the steel to the free shrinkage strain of the concrete, Miller, et al recommends using a ratio of 0.1 for heavily reinforced concrete and 0.3 for moderately reinforced concrete.

The method is not intended for use with decks with two layers of reinforcing steel. However, if the method is modified and applied to the deck assuming the same values derived using the effective modulus method used in Section 6.4.4, then the effective strain at the interface of the deck and the girder can be estimated. Using a free shrinkage strain of 143 $\mu \varepsilon$, a final shrinkage strain of 127 $\mu \varepsilon$ for the steel, and the distance $d$ to the center of the two layers of reinforcing steel of 4.0 in., the curvature can be calculated as 4.0 $\mu \varepsilon$ and the effective shrinkage strain at the bottom of the bolster of 107 $\mu \varepsilon$. This behavior would reduce the effective strain causing rotation from 143 $\mu \varepsilon$ to 107 $\mu \varepsilon$. A reduction in effective shrinkage at the top of the girder would help to explain why the predicted restraint moments are much higher than observed at the later days of 28 and 20.

Without having strain gages vertically throughout the deck, there is no way of knowing whether this type of behavior actually occurred during the testing. Also, assuming this type of behavior allows the curvature in the deck and the girder to be different while most conventional design methods require the curvatures to be the same.

**Extensibility of the Deck Concrete** – According to Mehta, extensibility can be defined as the ability of a concrete to deform in tension without cracking (Mehta 1986). A concrete with a low modulus of elasticity generally has better extensibility than a concrete with a high modulus of elasticity. At the early age when a majority of the shrinkage is occurring, the modulus of elasticity is also lower. From Hooke’s Law, a lower modulus of elasticity also produces lower stress for a given strain. When shrinkage is occurring at early ages, the actual tensile stresses produced by the strains are likely to be less because of a combination of the lower modulus of
elasticity and the extensibility of the concrete.

Considering applied forces, if a tensile force is applied to a concrete cylinder instantly to produce failure in tension, then the tensile strain required to cause failure can be determined. However, if the load is applied very slowly, then the strain at failure can be expected to be 1.2 to 2.5 times the strain at failure due to the suddenly applied load (Branson, 1977). The second cylinder is said to have better extensibility than the first.

Since shrinkage happens early when the modulus of elasticity is low and the stresses caused by shrinkage are slowly applied, it is possibly that effective forces caused by shrinkage are lower than conventionally predicted forces because of the extensibility of the concrete. For a deck placed on a girder, the shrinkage of the deck is conventionally assumed to cause the top of the girder to go into compression and the bottom of the deck to go into tension. Extensibility of the concrete may cause the amount of compression in the girder to be less than conventionally predicted. The strains in the deck due to the differential shrinkage are unusual, because the driving force, the shrinkage, wants to cause compressive shrinkage strains in the girder, but the resistance of the girder wants to cause tensile expansive strains in the deck. When exposed to free shrinkage, no stresses are induced because of shrinkage strains. However, when placed on a girder, the net tensile strains produce stresses, causing tension in the deck and possibly allowing for a reduction in the stresses produced because of the extensibility of the concrete.

The concept of extensibility is similar to creep in tension. But, a fundamental difference occurs in the case of differential shrinkage. The shrinkage does not occur because of an externally applied load. Instead, internal forces due to mechanisms such as the loss of adsorbed water cause movements to occur. Equating the equivalent movement caused by an external force to the movement caused by shrinkage may not necessarily be completely valid.
7 Service and Cyclic Phase Results and Discussions

7.1 Overview of the Tests Performed

Table 7.1 shows a summary of the tests performed for Test 1 and Test 2 and Table 7.2 shows a summary of the tests performed for Test 3. Load Control was used for all tests in Test 1 and 2. The Range column shows the range of the applied moment as a fraction of the predicted cracking moment, Mcr, calculated with the gross section properties at the interface and the typically used modulus of rupture of 7.5 times the square root of the compressive strength. A value of 0, 0.75, 0 indicates that the active end load is adjusted to apply zero moment at the continuity connection, then gradually increased to 0.75Mcr, and then gradually decreased to zero moment again. The Number of Cycles column shows the number of cycles of thermal restraint moments applied prior to performing a static test of 0, 1.0Mcr, 0. The magnitude of the thermal restraint moments is increased as described in the Notes column. The Interface Crack Width column shows the range of the manually measured crack widths using a crack comparator card near the bottom of the girder and diaphragm interface. This value is similar in magnitude to the value measured with the LVDTs at the same location. For Test 3, Load Control was used for Tests A and B and Deflection Control was used for the remaining tests. The Range column shows both the applied moment and the active end deflection measured by the MTS System. The Number of Cycles Column shows the number of deflection controlled cycles ranging from an active end deflection of 0.7 in. to -0.2 in. for negative moment and ranging from 0.7 in. to 1.6 in. for positive moment. The Maximum Top of Deck Crack Width Column shows the visually measured crack widths along the top of the deck.

7.2 Service Load Testing

For Test 1 and Test 2, test series A through C were performed to investigate the response of the structure to initial service loads prior to the application of any cyclic loads. For Test 3, test series A and B were performed to investigate the same response. As the loads were slowly applied, the structure was visually monitored to determine when first cracking of the interface of the diaphragm and the girder occurred due to positive moment. The electronic data was manually recorded with the System 5000 at predetermined magnitudes of applied moment. All loads were applied as a fraction of the predicted cracking moment, Mcr, and are presented as such.
### Table 7.1 – Summary of Static and Cyclic Tests for Test 1 and Test 2

<table>
<thead>
<tr>
<th>Test Name</th>
<th>Range, %Mcr</th>
<th>Number of Cycles</th>
<th>Interface Crack Width, in.</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>0, 0.75, 0</td>
<td>0</td>
<td>0.007</td>
<td>Initial cracking at interface prior to 0.5Mcr, 0.002 in. max</td>
</tr>
<tr>
<td>1B</td>
<td>0, 1.0, 0</td>
<td>0</td>
<td>0.010 - 0.013</td>
<td>Interface cracking returns to 0.002 in.</td>
</tr>
<tr>
<td>1C</td>
<td>0, 1.20, 0</td>
<td>0</td>
<td>0.016 - 0.025</td>
<td>Initial cracking in edge of deck</td>
</tr>
<tr>
<td>1D</td>
<td>0, 1.0, 0</td>
<td>1,000</td>
<td>0.020 - 0.025</td>
<td>Cycles at 0.25 Mthermal</td>
</tr>
<tr>
<td>1E</td>
<td>0, 1.0, 0</td>
<td>2,000</td>
<td>0.020 - 0.025</td>
<td>Cycles at 0.50 Mthermal</td>
</tr>
<tr>
<td>1F</td>
<td>0, 1.0, 0</td>
<td>2,000</td>
<td>0.020 - 0.025</td>
<td>Cycles at 0.75 Mthermal, minor spalling noticed</td>
</tr>
<tr>
<td>1G</td>
<td>0, 1.0, 0</td>
<td>2,000</td>
<td>0.030 - 0.040</td>
<td>Cycles at 1.00 Mthermal, one reading up to 0.050 in.</td>
</tr>
<tr>
<td>1H</td>
<td>0, 1.0, 0</td>
<td>2,000</td>
<td>0.030 - 0.040</td>
<td>Cycles at 1.00 Mthermal, concrete dusting and spalling</td>
</tr>
<tr>
<td>1J</td>
<td>0, 1.0, 0</td>
<td>1,000</td>
<td>0.040 - 0.050</td>
<td>Cycles at 1.00 Mthermal, cracks remain 0.007 to 0.010 in.</td>
</tr>
<tr>
<td>1K</td>
<td>0, 1.0, 0</td>
<td>2,000</td>
<td>0.040 - 0.050</td>
<td>Additional cracking in deck up to 0.010 in.</td>
</tr>
<tr>
<td>1L</td>
<td>0, 1.0, 0,-1,0,1.0,-1,0</td>
<td>0</td>
<td>0.040 - 0.050</td>
<td>Cracks in deck and at interface not completely closed</td>
</tr>
<tr>
<td>1M</td>
<td>0, 1.0, 0,-1,0,1.7,0</td>
<td>0</td>
<td>0.105</td>
<td>Initial cracking in bottom of diaphragm at 1.56 Mcr</td>
</tr>
<tr>
<td>1N</td>
<td>0, 1.83, 0</td>
<td>0</td>
<td>0.150</td>
<td>Continuous record to near failure</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test Name</th>
<th>Range, %Mcr</th>
<th>Number of Cycles</th>
<th>Interface Crack Width, in.</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>2A</td>
<td>0, 0.75, 0</td>
<td>0</td>
<td>0.005 - 0.010</td>
<td>Initial cracking at interface prior to 0.5Mcr, 0.005 in. max</td>
</tr>
<tr>
<td>2B</td>
<td>0, 1.0, 0</td>
<td>0</td>
<td>0.009 - 0.016</td>
<td>Interface cracking returns to 0.002 to 0.003 in.</td>
</tr>
<tr>
<td>2C</td>
<td>0, 1.20, 0</td>
<td>0</td>
<td>0.016 - 0.025</td>
<td>Cracking extends half way up web, none in deck</td>
</tr>
<tr>
<td>2D</td>
<td>0, 1.0, 0</td>
<td>1,000</td>
<td>0.010 -0.020</td>
<td>Cycles at 0.25 Mthermal</td>
</tr>
<tr>
<td>2E</td>
<td>0, 1.0, 0</td>
<td>2,000</td>
<td>0.016 - 0.020</td>
<td>Cycles at 0.50 Mthermal, cracking extends to top of girder</td>
</tr>
<tr>
<td>2F</td>
<td>0, 1.0, 0</td>
<td>2,000</td>
<td>0.016 - 0.025</td>
<td>Cycles at 0.75 Mthermal, minor spalling noticed</td>
</tr>
<tr>
<td>2G</td>
<td>0, 1.0, 0</td>
<td>2,000</td>
<td>0.016 - 0.025</td>
<td>Cycles at 1.00 Mthermal, minor cracking on side of deck</td>
</tr>
<tr>
<td>2H</td>
<td>0, 1.0, 0</td>
<td>2,000</td>
<td>0.016 - 0.025</td>
<td>Cycles at 1.00 Mthermal, minor cracking on web fillet</td>
</tr>
<tr>
<td>2J</td>
<td>0, 1.0, 0</td>
<td>1,000</td>
<td>0.016 - 0.025</td>
<td>Cycles at 1.00 Mthermal, cracks remain 0.007 to 0.009 in.</td>
</tr>
<tr>
<td>2K</td>
<td>0, 1.0,-1,0</td>
<td>0</td>
<td>0.016 - 0.025</td>
<td>Cracking on edge of deck 0.013 in.</td>
</tr>
<tr>
<td>2L</td>
<td>0, 1.0,-1,0,1.0,-1,0,1.0,-1,0,1.0,-1,0</td>
<td>0</td>
<td>0.016 - 0.025</td>
<td>Maximum crack on deck 0.016 in.</td>
</tr>
<tr>
<td>2M</td>
<td>0, 2.07, 0</td>
<td>0</td>
<td>0.130</td>
<td>Continuous record to near failure</td>
</tr>
</tbody>
</table>

### Table 7.2 – Summary of Static and Cyclic Tests for Test 3

<table>
<thead>
<tr>
<th>Test Name</th>
<th>Range, Moment (kft) and Deflection (in.)</th>
<th>Number of Cycles</th>
<th>Max. Top of Deck Crack Width, in.</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>3A</td>
<td>Moment -0.1 -73.1 -0.6 68.1 -0.1</td>
<td>0</td>
<td>0.016 - 0.020</td>
<td>Initial Cracking at -17.5 kft</td>
</tr>
<tr>
<td>3B</td>
<td>Moment -0.3 -71.4 -0.5 68.6 -0.2</td>
<td>0</td>
<td>0.020 - 0.040</td>
<td>Max. Bottom Cracking of 0.025 in.</td>
</tr>
<tr>
<td>3C</td>
<td>Moment 23.6 -50.4 20.4 62.6 16.4</td>
<td>2,000</td>
<td>0.016 - 0.030</td>
<td>Max. Bottom Cracking of 0.025 in.</td>
</tr>
<tr>
<td>3D</td>
<td>Moment 17.2 -51.6 19.5 60.7 15.2</td>
<td>2,000</td>
<td>0.016 - 0.040</td>
<td>Max. Bottom Cracking of 0.030 in.</td>
</tr>
<tr>
<td>3E</td>
<td>Moment 18.2 -51.0 18.2 60.4 15.2</td>
<td>2,000</td>
<td>0.020 - 0.040</td>
<td>Max. Bottom Cracking of 0.025 in.</td>
</tr>
<tr>
<td>3F</td>
<td>Moment 15.5 -50.8 18.8 59.8 15.1</td>
<td>2,000</td>
<td>0.016 - 0.040</td>
<td>Max. Bottom Cracking of 0.030 in.</td>
</tr>
<tr>
<td>3G</td>
<td>Moment 17.0 -49.8 19.5 60.7 15.3</td>
<td>2,000</td>
<td>0.016 - 0.040</td>
<td>Max. Bottom Cracking of 0.030 in.</td>
</tr>
<tr>
<td>3H</td>
<td>Moment 18.7 -47.2 21.3 60.9 17.6</td>
<td>0</td>
<td>0.016 - 0.040</td>
<td>Max. Bottom Cracking of 0.030 in.</td>
</tr>
<tr>
<td>3J</td>
<td>Moment 19.1 -48.1 23.3 64.3 17.8</td>
<td>0</td>
<td>0.100 (+)</td>
<td>Max. Bottom Cracking of 0.100 (+) in.</td>
</tr>
</tbody>
</table>
7.2.1 Initial Cracking Moment

Current design methods recommend using the initial cracking moment capacity of the positive moment connection, $M_{cr}$, to be based on the gross section properties of the girder, bolster, and deck at the interface and the tensile capacity of the concrete equal to the modulus of rupture of the diaphragm concrete, which is $7.5 \times \sqrt{f'_c}$ times the square root of the compressive strength of the diaphragm concrete. Since the diaphragm concrete is placed against the end of the already cured girder, it was suspected that the modulus of rupture would overestimate the actual tensile capacity of the concrete at the interface. The results of the initial cracking moment tests for all three tests confirm that the initial cracking does occur at a calculated tensile stress of the concrete lower than the modulus of rupture.

Table 7.3 shows the First Cracking moments observed (when first cracking was noticed at the bottom of the girder/diaphragm interface) and predicted for all three tests. The predicted tensile capacity of the concrete is presented two ways for five different methods. The first set of predicted values is based on the As Designed value of the tensile capacity of the concrete based on the 28 day compressive strength of 4,000 psi. The second set of predicted values is based on the As Observed values of compressive strength and modulus of rupture determined from the cylinder testing. Method 1 uses the currently recommended value of the modulus of rupture, $7.5(f'_c)^{1/2}$. Method 2 uses the ACI code recommended low range for direct tensile strength equal to $0.1f'_c$ (ACI Building 2002). Method 3 uses a recommended formula for direct tensile strength as a function of the unit weight of the concrete (assumed to be 145 pcf) and the compressive strength of the concrete as proposed by Branson, $(1/3)(w_f c)(1/2)$ (Branson 1977). Method 4 uses a relationship proposed by Mehta for direct tension as a function of the modulus of rupture, $0.59f_r$ (Mehta 1986). Method 5 is a proposed method based on the Branson method with the fraction $(1/3)$ reduced to $(1/4)$.

As shown in Table 7.3, the ratio of the Designed Initial Cracking Moment to the Observed Cracking Moment is calculated for all three tests. Values closest to unity indicate the best prediction of the initial cracking moment. For all three tests, the currently used method of predicting initial cracking moment, Method 1, over predicts the actual capacity of the initial cracking moment by a significant amount, with ratios ranging from 1.52 to 3.29. For Test 1 and 2, Method 5 is the best at predicting initial cracking moment with ratios of 0.90 to 1.19. For Test 3, the cracking moments presented are the cracking moments of the deck which is continuous.
over the supports. Since the deck is continuous, it is expected to initially crack at a higher tensile stress than the sections in Test 1 and Test 2 which are cast against the ends of the girders. This is shown in the table where Method 5 under predicts the capacity by a factor of 0.61. Method 4 provides the best predictor for the initial cracking with ratios of 0.90 and 0.93.

Table 7.3 – Summary of Initial Cracking

<table>
<thead>
<tr>
<th>Test Number 1 - First Cracking</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>ft as Designed, psi</td>
<td>474</td>
<td>400</td>
<td>253</td>
<td>280</td>
<td>190</td>
</tr>
<tr>
<td>MCr as Designed, Kft</td>
<td>531.38</td>
<td>448.10</td>
<td>283.40</td>
<td>313.51</td>
<td>212.55</td>
</tr>
<tr>
<td>MCr Observed</td>
<td>214.76</td>
<td>214.76</td>
<td>214.76</td>
<td>214.76</td>
<td>214.76</td>
</tr>
<tr>
<td>Designed/Observed</td>
<td>2.47</td>
<td>2.09</td>
<td>1.32</td>
<td>1.46</td>
<td>0.99</td>
</tr>
<tr>
<td>ft as observed, psi</td>
<td>630</td>
<td>579</td>
<td>304</td>
<td>372</td>
<td>228</td>
</tr>
<tr>
<td>MCr as Designed, Kft</td>
<td>705.75</td>
<td>648.62</td>
<td>340.97</td>
<td>416.39</td>
<td>255.72</td>
</tr>
<tr>
<td>MCr Observed</td>
<td>214.76</td>
<td>214.76</td>
<td>214.76</td>
<td>214.76</td>
<td>214.76</td>
</tr>
<tr>
<td>Designed/Observed</td>
<td>3.29</td>
<td>3.02</td>
<td>1.59</td>
<td>1.94</td>
<td>1.19</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test Number 2 - First Cracking</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>ft as Designed, psi</td>
<td>474</td>
<td>400</td>
<td>253</td>
<td>280</td>
<td>190</td>
</tr>
<tr>
<td>MCr as Designed, Kft</td>
<td>531.38</td>
<td>448.10</td>
<td>283.40</td>
<td>313.51</td>
<td>212.55</td>
</tr>
<tr>
<td>MCr Observed</td>
<td>214.76</td>
<td>214.76</td>
<td>214.76</td>
<td>214.76</td>
<td>214.76</td>
</tr>
<tr>
<td>Designed/Observed</td>
<td>2.25</td>
<td>1.90</td>
<td>1.20</td>
<td>1.33</td>
<td>0.90</td>
</tr>
<tr>
<td>ft as observed, psi</td>
<td>580</td>
<td>584</td>
<td>306</td>
<td>342</td>
<td>229</td>
</tr>
<tr>
<td>MCr as Designed, Kft</td>
<td>649.74</td>
<td>654.22</td>
<td>342.44</td>
<td>383.35</td>
<td>256.83</td>
</tr>
<tr>
<td>MCr Observed</td>
<td>214.76</td>
<td>214.76</td>
<td>214.76</td>
<td>214.76</td>
<td>214.76</td>
</tr>
<tr>
<td>Designed/Observed</td>
<td>2.76</td>
<td>2.78</td>
<td>1.45</td>
<td>1.63</td>
<td>1.09</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test Number 3 - First Cracking</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>ft as Designed, psi</td>
<td>474</td>
<td>400</td>
<td>253</td>
<td>280</td>
<td>190</td>
</tr>
<tr>
<td>MCr as Designed, Kft</td>
<td>-26.68</td>
<td>-22.50</td>
<td>-14.23</td>
<td>-15.74</td>
<td>-10.67</td>
</tr>
<tr>
<td>MCr Observed</td>
<td>-17.5</td>
<td>-17.5</td>
<td>-17.5</td>
<td>-17.5</td>
<td>-17.5</td>
</tr>
<tr>
<td>Designed/Observed</td>
<td>1.52</td>
<td>1.29</td>
<td>0.81</td>
<td>0.90</td>
<td>0.61</td>
</tr>
<tr>
<td>ft as observed, psi</td>
<td>490</td>
<td>402</td>
<td>254</td>
<td>289</td>
<td>190</td>
</tr>
<tr>
<td>MCr as Designed, Kft</td>
<td>-27.56</td>
<td>-22.61</td>
<td>-14.26</td>
<td>-16.26</td>
<td>-10.70</td>
</tr>
<tr>
<td>MCr Observed</td>
<td>-17.5</td>
<td>-17.5</td>
<td>-17.5</td>
<td>-17.5</td>
<td>-17.5</td>
</tr>
<tr>
<td>Designed/Observed</td>
<td>1.57</td>
<td>1.29</td>
<td>0.82</td>
<td>0.93</td>
<td>0.61</td>
</tr>
</tbody>
</table>
Figure 7.1 shows the initial cracking of 0.016 in. along the interface of the girder and the diaphragm for Test 2B at the loading of 1.0Mcr. The initial cracking of Test 1 and Test 2 at the interface was well defined for both tests. Since the interface at the ends of the girders acts as a cold joint (no bonding agent was used), the cracking can be considered as debonding of the diaphragm concrete from the end of the girder. Although the debonding does produce an opening that is large enough to allow the intrusion of water into the section, the crack caused by the debonding can be mitigated with a sealant since the debonding is well defined.

For prediction of the initial positive moment cracking moment at the interface of the girder and the diaphragm, it is recommended that the tensile capacity of the concrete at the bottom of the connection be estimated with Method 5. Gross section properties considering the full profile of the end of the girder, the bolster, and the full width of the deck should be used for determining the cracking moment capacity. For the initial cracking of the deck, the capacity of the deck to resist moments across the opening between the ends of the girders is generally not considered in the design. The predicted capacity using Method 4 is best at predicting the cracking moment observed, but since the loads are so low across the joint, this predicted moment will generally not be used in design.
7.2.2 Initial Loading Behavior

Figure 7.2 shows the initial loading behavior for tests A through C for Test 1 and Test 2. The percent of the applied cracking moment, Mcr, is plotted versus the average LVDT reading at the bottom of the girder and diaphragm interface. The average LVDT reading gives approximately the same value as the average measured crack width using a crack comparator card. The response of both tests to the applied positive moments is similar. Test 2, with the 180 degree U-bars, displays a slightly stiffer response to the application of the positive moment. For an applied moment of 1.2Mcr, the average opening as measured by the LVDTs is 0.022 in. versus an average opening of 0.029 in. for Test 1. Based on the ultimate predicted capacities, this behavior is expected. Test 1, with the strands, has a predicted ultimate positive moment capacity of 711.4 k-ft and Test 2, with the bars, has a predicted ultimate positive moment capacity of 773.3 k-ft, giving a ratio of 0.92. The calculations for these capacities are included in Appendix F. The ratio of the average LVDT readings is also less than one, but even lower at 0.76. Therefore, Test 2 is slightly stiffer than Test 1 for the initial loading phase. The ultimate calculated capacity of Test 1 is based on a reduced allowable stress in the prestressing strands at ultimate loading. Based on the embedment of the 90 degree bent strands in the diaphragm, the predicted stress at ultimate is 98.2 ksi. This is much lower than the ultimate breaking strength of a strand, which is generally taken as 270 ksi. The design procedure, shown in Section 2.3, has considered the fact that the strands are not able to reach the full tensile capacity of a fully developed tensioned strand (Salmons 1974).

All of the initial tests are similar in shape both during loading and unloading. A simplified rheological model is developed to help explain the behavior shown (Dowling 1993). Since cracking occurs early in Test A for both Test 1 and Test 2, the tensile capacity of the concrete at the interface does not contribute much to the moment resistance of the section. A moment is established at the interface with a compressive force located in the deck and a tensile force located at the approximate location of the tensile steel in the bottom of the section. During the service loading, the moment arm does not change much, since the location of the tensile force is more or less fixed at the location of the steel and the location of the compressive force remains near the top of the deck under service loads. This means that the moment capacity is primarily a function of the magnitude of the tensile and compressive forces. Considering the tensile force in the steel, the equivalent force in the steel is equal to the equivalent stiffness of the steel times the
Figure 7.2 – Initial Service Tests for Test 1 and Test 2
crack opening at the location of the steel. This is the basic stiffness relationship \( f=kd \). Dividing both sides of this relationship by the area of the steel gives the basic stress strain relationship: stress, \( \sigma \), is equal to the modulus of elasticity, \( E \), times the strain, \( \varepsilon \), (\( \sigma=E \varepsilon \)).

The rheological model proposed in Figure 7.3 describes the effective stress strain relationship of the steel in the bottom of the section at the interface. The effective modulus of elasticity, \( E_e \), is determined by:

\[
E_e = \frac{E_1E_2}{E_1 + E_2}
\]  

(7.1)

where \( E_1 \) is the initial slope of the stress strain diagram and \( E_2 \) is the slope that occurs in parallel with the slider mechanism after some stress level \( \sigma_o \) is reached. The slider mechanism prevents \( E_2 \) from acting until a stress of \( \sigma_o \) is reached. After reaching this level, part of the stress is carried by the slider and part of the stress is carried by the spring \( E_2 \). When unloaded, the slider mechanism will prevent spring \( E_2 \) from engaging until a stress of \( 2 \sigma_o \) is induced in the system. The initial slope, \( E_1 \), is higher at lower stress levels because as the crack begins to open, aggregate to aggregate interlock friction helps to resist the opening of the section and provides more tensile capacity. This aggregate to aggregate friction occurs until some level of stress \( \sigma_o \) is reached. After this point, as the crack width increases, the steel becomes the only resistance to tensile forces, shown as a reduced equivalent modulus of elasticity, \( E_e \). Upon unloading, the initial slope \( E_1 \) is followed until the slider mechanism, the aggregate to aggregate interlock, is forced back into compression. This helps to explain why the unloading line does not follow the loading line and why the curve does not return directly to zero.
Figure 7.4 – Initial Service Tests for Test 3
Figure 7.4 above shows the behavior of Test 3 for tests A and B. The applied moment is plotted versus the active end deflection and the LVDT readings near the bottom of the section and the top of the web at the joint location. The section was slowly loaded and the formation of cracks documented. The response of the structure to cyclic loads including the increase in crack widths and deck damage is discussed in Section 7.3.

Figure 7.5 shows that under the initial service loads, two primary cracks formed through the deck on each side of the 3 in. joint. The cracks are shown marked in black to the right of the sign. The cracks were approximately 9 in. apart on the outside edge of the deck and slightly closer together at the interior of the deck. It was expected that one joint may form instead of two. However, the initial service testing indicates that two relief joints instead of one should be used above the joint opening. The two relief joints should be centered above the joint and spaced at the maximum of (A) the slab thickness or (B) the joint opening plus two times the bolster thickness. For this test, the relief joints would be placed at 7.5 in., the maximum of (A) 7.5 in. or (B) 3 + 2(1.5) = 6 in.

Figure 7.5 – Initial Cracking in Deck, Test 3

7.2.3 Prediction of Initial Crack Widths

Figure 7.6 presents a simplified model to predict the initial crack widths at the interface of the end of the girder and the diaphragm for Test 1 and Test 2. The model assumes that once first cracking occurs, cracked section properties can be used to determine the strain in the steel, $\varepsilon_a$, at the crack opening. The strain in the concrete at the crack opening is assumed to be zero. At the end of the embedded strand or bar in the continuity diaphragm, a distance $f$ as shown, uncracked transformed section properties are used to determine the strain in the steel and the concrete which will be equal, $\varepsilon_b$. Assuming that the load on the bar is symmetric about the
crack, half of the predicted crack opening, \( w \), can then be determined by integrating the difference in the strain in the steel and the strain in the concrete over the distance, \( f \) as:

\[
\frac{w}{2} = \int_{0}^{f} (\varepsilon_s - \varepsilon_c) \, df
\]  

(7.2)

where \( \varepsilon_s \) is an expression for the strain in the steel and \( \varepsilon_c \) is an expression for the strain in the concrete. For the expressions shown, the crack width reduces to the simple formula of:

\[
w = f \varepsilon_a
\]  

(7.3)

For the initial service load test, Test A, for Test 1 with the strands, the model predicts a crack opening of 0.016 in. and the observed average LVDT reading was 0.012 in., similar in magnitude but over predicted by a ratio of 1.30. For Test 2, the model predicts a crack opening of 0.011 in. and the observed average LVDT reading was 0.009 in., also similar in magnitude but over predicted by a ratio of 1.20.

Although simplified, the model does reasonably well at predicting the average opening at the moment causing initial cracking. This model is also used to predict crack widths due to positive moments applied during the cyclic testing in the following section where the calculations are provided.
7.3 Cyclic Load Testing

The purpose of the Cyclic Load Testing phase was to determine the ability of the three continuity details to withstand moments due to expected thermal cycles. In order to avoid overloading the specimens at early loads, the magnitudes of the moments applied were gradually increased as shown in Table 7.1.

7.3.1 Behavior During Cyclic Loads

Figure 7.7 shows the cyclic tests D through J for Test 1 and Test 2. The percent of the cracking moment is plotted versus the average LVDT opening at the bottom of the interface. The results are presented at the same scale so that a visual comparison can be easily made between the two. Both tests maintained integrity throughout the cyclic loading phase. Although both specimens were able to withstand the 10,000 thermal cycles and continue to carry loads, it is apparent that Test 2 with the embedded bars performed better than Test 1 with the strands. After the final 10,000 cycles had been applied, Test 1 displayed an average LVDT opening of 0.047 in. while Test 2 displayed an average LVDT opening of only 0.018 in., 2.6 times less opening. This indicates that Test 2 was stiffer under the cyclic loads.

The behavior of Test 1 changed throughout the duration of the cyclic loads. Figure 7.8 shows a modification of the rheological model proposed to explain the change in behavior. During cyclic loads, untensioned prestressing strands have been known to develop a tendency to slip (Salmons 1974). As the strands are pulled in tension, the wires have a tendency to twist and locally unwind through the surrounding concrete, causing some localized crushing. For a tensioned strand, Hoyer’s effect acts to counteract the loss in development due to the twisting action. However, for untensioned strands, such as are in the Test 1 continuity diaphragm, Hoyer’s effect does not help to increase the bond stresses. Throughout the cycles, the strands develop a much lower bond from the location of the interface to the 90 degree bend. The reduced bond can be modeled as an initial modulus of elasticity, $E_3$, that occurs for a distance $d$ until the previous model is engaged.

The response of Test 2 to the cyclic loads remains virtually unchanged throughout the testing. Following the service load testing, the role of the aggregate to aggregate friction in increasing the tensile force capacity is diminished as the tensile steel becomes the dominate provider of tensile capacity.
Figure 7.7 – Cyclic Tests for Test 1 and Test 2
7.3.2 Increase in Crack Openings due to Cyclic Loads

It is typical to compare the stress to number of cycles, or S-N curve, for a test performed with cyclic loads. For Test 1 and 2, the applied moment at testing was kept the same, so a comparison of stress does not provide any useful information. Instead, a comparison is made showing the average LVDT reading, or average crack opening, at the bottom of the interface versus the number of cycles shown on a log scale. Figure 7.9 shows this comparison.

The graph shows that over the 10,000 cycles, the crack opening for Test 2 increases only slightly and remains fairly linear throughout the life of the testing. However, for Test 1, the crack opening starts off linear, at nearly the same slope as Test 2, but at approximately 5000 cycles, begins to increase rapidly. This type of behavior is undesirable, indicating that the system is softening during the cyclic loads. Although the connection is still able to transfer the moment across the connection, this behavior indicates that the system has a decrease in continuity during the loading. For a given moment, more rotation occurs at the connection.

Using the previously described model for the prediction of crack widths, Figure 7.10 shows a comparison of the ratio of the predicted crack opening to the average crack opening measured at the bottom of the interface with the LVDTs. The ratios are plotted versus the number of cycles. A perfect prediction is indicated with a ratio of 1.0.
Comparison of Crack Openings

Figure 7.9 – Opening versus Life Comparison for Test 1 and Test 2

Figure 7.10 – Predicted versus Observed Crack Openings
The model for predicting the crack opening through the cyclic phase does reasonably well for Test 2, but under predicts the crack openings for Test 1 as the number of cycles is increased. The decreased bond of the prestressing strand causes the assumptions of strains along the length of the strand to become invalid. Table 7.4 shows the calculations for the predicted crack openings and the ratio of the predicted to observed crack openings.

Table 7.4 – Calculations for Crack Opening Widths

<table>
<thead>
<tr>
<th>Test</th>
<th>M applied k-ft</th>
<th>No. Cycles</th>
<th>Strand Stress, $f_s$ ksi</th>
<th>Strain, $\varepsilon_s \mu \varepsilon$</th>
<th>Observed Opening in.</th>
<th>Predicted Opening in.</th>
<th>Ratio Pred./Obs.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-A</td>
<td>392.9</td>
<td>1</td>
<td>50.6</td>
<td>1733</td>
<td>0.012</td>
<td>0.016</td>
<td>1.30</td>
</tr>
<tr>
<td>1-B</td>
<td>523.8</td>
<td>2</td>
<td>67.5</td>
<td>2310</td>
<td>0.018</td>
<td>0.021</td>
<td>1.16</td>
</tr>
<tr>
<td>1-C</td>
<td>628.6</td>
<td>3</td>
<td>81.0</td>
<td>2772</td>
<td>0.029</td>
<td>0.025</td>
<td>0.86</td>
</tr>
<tr>
<td>1-D</td>
<td>523.8</td>
<td>1000</td>
<td>67.5</td>
<td>2310</td>
<td>0.023</td>
<td>0.021</td>
<td>0.90</td>
</tr>
<tr>
<td>1-E</td>
<td>529.0</td>
<td>3000</td>
<td>68.1</td>
<td>2333</td>
<td>0.024</td>
<td>0.021</td>
<td>0.88</td>
</tr>
<tr>
<td>1-F</td>
<td>523.8</td>
<td>5000</td>
<td>67.5</td>
<td>2310</td>
<td>0.027</td>
<td>0.021</td>
<td>0.77</td>
</tr>
<tr>
<td>1-G</td>
<td>523.8</td>
<td>7000</td>
<td>67.5</td>
<td>2310</td>
<td>0.040</td>
<td>0.021</td>
<td>0.52</td>
</tr>
<tr>
<td>1-H</td>
<td>523.8</td>
<td>9000</td>
<td>67.5</td>
<td>2310</td>
<td>0.045</td>
<td>0.021</td>
<td>0.46</td>
</tr>
<tr>
<td>1-J</td>
<td>523.8</td>
<td>10000</td>
<td>67.5</td>
<td>2310</td>
<td>0.047</td>
<td>0.021</td>
<td>0.44</td>
</tr>
<tr>
<td>2-A</td>
<td>392.9</td>
<td>1</td>
<td>28.2</td>
<td>1017</td>
<td>0.009</td>
<td>0.011</td>
<td>1.22</td>
</tr>
<tr>
<td>2-B</td>
<td>529.0</td>
<td>2</td>
<td>37.9</td>
<td>1369</td>
<td>0.014</td>
<td>0.015</td>
<td>1.09</td>
</tr>
<tr>
<td>2-C</td>
<td>628.6</td>
<td>3</td>
<td>45.1</td>
<td>1626</td>
<td>0.020</td>
<td>0.018</td>
<td>0.88</td>
</tr>
<tr>
<td>2-D</td>
<td>523.8</td>
<td>1000</td>
<td>37.5</td>
<td>1355</td>
<td>0.016</td>
<td>0.015</td>
<td>0.95</td>
</tr>
<tr>
<td>2-E</td>
<td>523.8</td>
<td>3000</td>
<td>37.5</td>
<td>1355</td>
<td>0.016</td>
<td>0.015</td>
<td>0.95</td>
</tr>
<tr>
<td>2-F</td>
<td>523.8</td>
<td>5000</td>
<td>37.5</td>
<td>1355</td>
<td>0.017</td>
<td>0.015</td>
<td>0.90</td>
</tr>
<tr>
<td>2-G</td>
<td>523.8</td>
<td>7000</td>
<td>37.5</td>
<td>1355</td>
<td>0.017</td>
<td>0.015</td>
<td>0.90</td>
</tr>
<tr>
<td>2-H</td>
<td>523.8</td>
<td>9000</td>
<td>37.5</td>
<td>1355</td>
<td>0.017</td>
<td>0.015</td>
<td>0.90</td>
</tr>
<tr>
<td>2-J</td>
<td>529.0</td>
<td>10000</td>
<td>37.9</td>
<td>1369</td>
<td>0.017</td>
<td>0.015</td>
<td>0.91</td>
</tr>
</tbody>
</table>

Figure 7.11 shows the cyclic test behavior for test C and test G for Test 3. After the initial cracking which occurred during the service load phase, the behavior of Test 3 during the cyclic loads did not change appreciably. Therefore, only the initial and final cyclic test results are provided. Tests D, E, and F are shown in Appendix G. Most of the cracking in the deck occurred during the initial loads with only a minor increase in damage due to the thermal cycles.

**7.3.3 Increase in Deck Damage due to Cyclic Loads**

Table 7.5 shows a summary of the increase in deck damage due to the cyclic loads. Crack maps are presented in Appendix H. The top of the table shows the maximum observed
Figure 7.11 – Cyclic Test C and G for Test 3
Crack width following the testing phase indicated. For Test 1 and Test 2, no significant deck cracking was noticed after the service tests since the tests were performed with positive moment only. But for Test 3, which used both positive and negative moment for the service testing, significant deck cracking was observed. Test 1 and Test 2 showed an increase in the deck crack widths due to the thermal cycles while Test 3 did not exhibit a significant change. The bottom of the table shows the increase in deck damage as measured by the increase in the linear feet of cracking for Test 3. Most of the damage occurred in the first test. The final damage shows the total linear feet of cracks noted for all three tests after all testing, including the ultimate strength testing, was performed. The cracking for Test 1 and Test 2 occurred as much as 5.5 ft and 6.9 ft respectively out from the center of the test specimen indicating that the continuity connection is able to transfer significant negative moments away from the center. For Test 3, most of the cracking occurred near the center joint within 1 ft to 2 ft, indicating that once cracking occurred, the section does not transfer significant moments through the joint.

Figure 7.12 shows the final minor cracking in the deck for Test 1 and Test 2. The maximum measured crack widths in the deck were generally less than approximately 0.013 in. The current ACI Code does not specifically address crack widths, but the previous versions of the ACI Code recommended keeping crack widths less than 0.016 in. for interior exposure and 0.013 in. for exterior exposure. (MacGregor, 1988) Since the crack widths observed are at the limit for exterior exposure, precautions should be taken to prevent corrosion of the steel in the deck due to deicer salts.

Figure 7.13 shows the final cracking of the deck for Test 3. It is expected that two control joints placed above the 3 in. joint between the ends of the girders will help to reduce and

<table>
<thead>
<tr>
<th>Test</th>
<th>Service Tests</th>
<th>Cyclic Tests</th>
<th>Negative Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>&lt; 0.002</td>
<td>0.007</td>
<td>0.010</td>
</tr>
<tr>
<td></td>
<td>&lt; 0.002</td>
<td>0.013</td>
<td>0.013</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test</th>
<th>Increase In Damage, LF</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test 1</td>
</tr>
<tr>
<td>Static Phase</td>
<td>N/A</td>
</tr>
<tr>
<td>3A</td>
<td>N/A</td>
</tr>
<tr>
<td>3B</td>
<td>N/A</td>
</tr>
<tr>
<td>3C</td>
<td>N/A</td>
</tr>
<tr>
<td>3D</td>
<td>N/A</td>
</tr>
<tr>
<td>3E</td>
<td>N/A</td>
</tr>
<tr>
<td>3F</td>
<td>N/A</td>
</tr>
<tr>
<td>3G</td>
<td>N/A</td>
</tr>
<tr>
<td>Final Damage</td>
<td>73.7</td>
</tr>
</tbody>
</table>

Note: LF is Linear Feet of Cracking
control the magnitude and number of cracks in the area of the joint. Since the crack widths observed are as large as 0.040 in., extra precautions should also be taken to prevent corrosion of the steel in the deck due to deicer salts.

7.3.4 **Continuity**

Both continuity connections for Test 1 and Test 2 were able to resist the service loads applied during the Service and Cyclic Phases. Also, both connections were able to transfer the service loads (up to 1.2Mcr) through the connection throughout the testing. Both connections showed some softening during the initial service load testing. Throughout the cyclic load testing, Test 1, with the strands, required more end rotation to resist the moments than did Test 2, with the bars. What exactly constitutes continuity in a continuous system differs widely. From a load standpoint, both connections did not exhibit a decrease in continuity during the testing.
However, from a rotational standpoint, both sections did see a reduction in the continuity of the system during the testing because more rotation was required to transfer the same moment.

### 7.4 Negative Moment Testing

The purpose of the negative moment testing was to confirm the repeatability of the response of the structure after being subjected to full negative cracking moments. Figure 7.14 shows test L for Test 1 and Test 2. Both tests were performed by first applying a positive cracking moment and then applying a negative cracking moment. Two complete load cycles were performed for each test, the first cycle is designated “A” and the second cycle is designated “B”. Both tests display good repeatability from the first cycle to the second. Test 1 continues to display a range of loading with a low slope indicating that the strands are twisting through the diaphragm until the 90 degree bend is engaged. Test 2 displays a nearly linear response in both the positive and negative moment directions. Tests 1K and 2K provide similar responses and are not shown here but are included in Appendix G.

### 7.5 Ultimate Strength Testing

The purpose of the Ultimate Strength Testing was to confirm the predicted ultimate positive moment capacities. Test 1 and Test 2 were taken to a near failure positive moment. This point was determined by monitoring the end deflection and applied moment. When the deflection began to increase substantially without a substantial increase in moment, the tests were considered near failure. A complete failure was avoided because it was deemed too dangerous. Test 3 was taken to the maximum end deflection possible. The negative moment was determined when the collar beam nearly touched the floor and the positive moment was determined when the MTS ram began to bind because of the excessive end rotation.

Figure 7.15 shows the results for the ultimate strength testing for Test 1 and Test 2. Both Test 1 and Test 2 reached and exceeded the predicted ultimate positive moment capacities. For consistency, the capacities are presented as a fraction of the cracking moment, 523.8 k-ft. For Test 1, the ultimate capacity approached slightly over 1.8Mcr while the predicted ultimate capacity $\Phi M_n$ is 1.36Mcr. The ratio of the observed to predicted capacity is 1.32 for Test 1. For Test 2, the ultimate capacity approached slightly over 2.0Mcr while the predicted ultimate capacity $\Phi M_n$ is 1.48Mcr. The ratio of the observed to predicted capacity is 1.35 for Test 2. Therefore, both design methods for predicting ultimate positive moment capacity provide
Figure 7.14 – Negative Moment Testing for Test 1 and Test 2
Figure 7.15 – Ultimate Strength Tests for Test 1 and Test 2

Test 3 Ultimate Loading - Use for Shape of Curve Only

Figure 7.16 – Ultimate Strength Test for Test 3
conservative estimates for the ultimate moments, over predicting the capacity by factors of 1.32 and 1.35.

Figure 7.16 above shows the results for the ultimate strength testing of the deck in Test 3J. It is believed that significant binding of the MTS ram began to occur as the loads were increased. The binding occurred because the end rotation at the active end was restrained by the collar beams. Because of the binding effect, the actual loads applied at the center of the deck are most likely much lower than the moments indicated in the figure. Therefore, the figure should be used only for its shape, and not the magnitude of the loads.

Figure 7.17 shows the bottom of the diaphragm connection following the ultimate strength testing. The significant cracking which occurred during the ultimate strength testing indicates that substantial stresses were introduced in the bottom of the continuity diaphragm connection. The cracking is located primarily within the limits of the bottom flange of the girders. Similar crack patterns were noticed for the two, with slightly less cracking in Test 2 than in Test 1.

There was no noticeable cracking in the girders for Test 1 with the strands. For Test 2, there was some minor cracking in the girders near the bottom of the web and along the top of the bottom flange. The cracking was first noticed at the end of 9000 thermal cycles, and was less than 0.002 in. in width. After the ultimate strength test, the number of cracks increased, with cracks forming at all four corners of the end of the girders at the diaphragm. The cracks never
opened much more than 0.002 in. and extended a maximum of only 19 in. into the girder. Figure 7.18 shows the final cracking at two locations. Any cracking is undesirable, but the cracking at the ends of the girders is minor both in crack widths and lengths, and should not pose a significant problem.

Figure 7.18 – Cracking in end of Girders for Test 2

7.6 Resolution of Design Issues

7.6.1 Preparing Bent Strands at the Fabricator

In the design phase of the project, there was concern expressed about cutting and bending the prestressing strands. Fraying of the strands has been noticed at the fabricator when strands are cut under tension with a torch. The fabricator avoided this problem by first cutting the end strands longer than required for the end connection. For this test, the strands needed to extend 24 in., so the fabricator first cut at approximately 36 in. and then made a final cut with the strands not under tension at the 24 in. location. This eliminated the problem with fraying. The strands were bent at the fabricator prior to delivery. A discussion with the foreman indicated that there were no problems with bending the strands, and that the strands were cold bent (no torch was used for the bending process).

7.6.2 Placing Mild Steel at the Fabricators

There was also concern expressed about placing the U shaped mild steel used in Test 2. The steel was designed to be placed within the end of the girder allowing for the typical 2 in. on center grid of prestressing strands to remain at 2 in. on center. The bars were placed directly against the strands, or bundled, and tied to the strands. The end form was modified by cutting
slotted holes in a plywood section used on the bottom only. After casting the girders, the form was pulled straight back off of the end of the girder. The foreman indicated that the method worked well, and that a plywood section is generally used anyway if any type of steel is extended from the ends of the girder.

There was also concern that the end termination of the mild steel may cause cracking to occur at the termination location. The termination was designed to stagger from top to bottom, the top length being longer than the bottom length, as proposed in NCHRP 519. However, the bars were fabricated with same length legs, and were already installed. Even though the bars had same length legs, no cracking was noticed at the end of bar termination location. It is possible that cracking occurs at the end of the bar termination because there are not enough stirrups at the termination location. For this test, the stirrups were closely spaced at the end, which helps with any cracking in the end of the girders.

7.6.3 Embedment of Girders into Diaphragm

Many states use a detail where the ends of the girders are embedded typically from 3 in. to 6 in. into the diaphragm. This detail was not tested because of two reasons. First, some reports indicate that the embedment does not significantly increase the capacity of the section. Second, it is suspected that any cracking that occurs due to rotations at the interface will cause spalling of the section because of the end of the girder exerting forces on the edge wedges of concrete. The results of this study indicate that sufficient strength can be achieved without embedding the ends of the girders. Also, the cracking which occurs when the girders are not embedded is well defined and can be mitigated whereas cracking and spalling of embedded girder ends may be much more difficult to mitigate.

7.6.4 Modification of Center Stirrups for Girders

The Virginia Department of Transportation recently modified the center stirrups that extend from the top of the girder from two separate stirrups with 90 degree legs to one single U shaped stirrup. The modification was made to help eliminate a tripping hazard on the top of the girders. Although not specifically tested, the testing performed did not provide any indication that the modification reduced the performance of the section.


8 Conclusions and Recommendations

The conclusions for each phase of the study are presented, followed by the recommendations for the entire study. The conclusions and recommendations are based on the type of girder and slab arrangement studied, and should not be considered applicable to structure types that are significantly different.

8.1 Design Phase Conclusions

1. When compared to the actual cracking moment capacity of a section at the continuity diaphragm, the predicted positive thermal restraint moments can be significant for commonly used girder spacings and span lengths, ranging from about 0.7 to 1.3 times the cracking moment capacity.

2. When compared to the most commonly used current methods, the PCA Method generally predicts the most conservative positive restraint moments due to time dependent effects such as creep and shrinkage.

3. For typically used span and strand arrangements, as the span length decreases, the predicted positive restraint moment due to creep and shrinkage generally also decreases.

4. For very early ages of continuity, some predicted positive restraint moments due to creep and shrinkage are greater than 1.2 times the cracking moment capacity, Mcr, but usually only at early ages of continuity.

5. For ages of continuity greater than approximately 60 days, Comparison Method 1 and the RMCalc Method predict similar positive restraint moments due to creep and shrinkage.

6. For ages of continuity greater than approximately 90 days, most current methods predict that no positive restraint moment will develop due to time dependent effects such as creep and shrinkage.

8.2 Laboratory Phase Conclusion

8.2.1 Related to Materials

1. The ACI equation for predicting the modulus of rupture, \( f_r = 7.5(f_c)^{0.5} \), under predicts the modulus of rupture for the high strength girder concrete by an average of 180 psi but
predicts the modulus of rupture for the lower strength deck concrete well.

2. The ACI equation for predicting the modulus of elasticity, \( E = 57,000(f'_c)^{0.5} \), predicts the modulus of elasticity reasonable well for the high strength girder concrete and predicts the modulus of elasticity very well for the lower strength deck concrete.

3. For shrinkage specimens monitored from day 1, one of the two tests indicates that significant expansion occurs during the first week prior to the onset of shrinkage.

4. For shrinkage specimens monitored from day 7, one of the three tests shows shrinkage similar to the ACI-209 predicted shrinkage while the other two tests show significantly more shrinkage than predicted by ACI. The MC-90 under predicts the early age shrinkage for all three tests.

5. The coefficients of thermal expansion for both the girder and the deck concrete are lower than the recommended value of 5.8 \( \mu e/\degree F \) for typical concrete proposed by the vibrating wire gage manufacturer. For the girder, a value of 4.7 \( \mu e/\degree F \) was determined; for the deck, a value of 5.1 \( \mu e/\degree F \) was determined.

8.2.2 Related to Instrumentation

6. Vibrating wire gage readings corrected for temperature changes are sensitive to the differences in the actual values of the coefficients of thermal expansion for the concrete and the gage. The correction for temperature can introduce significant errors if the actual coefficients of thermal expansion for the concrete and the gage are not known.

7. Strain gages that are placed in series in an arm of a Wheatstone Bridge produce an output which is an average of the two gages instead of the sum of the two gages.

8.3 Fabrication Phase Conclusions

1. Prior to casting the girders, bed temperatures potentially change significantly due to the heating or cooling of the steel forms causing the strands to either elongate or contract and change the prestressing force prior to casting. In this study, the heating of the forms may have caused a prestress loss of 3.6 ksi prior to casting the concrete.

2. Actual bed temperatures during steam curing may be much higher than believed by the
fabricator. Recorded bed temperatures for this study are nearly 40º F higher than recorded by the fabricator.

3. Considerable variation in the bed temperatures during steam curing may exist because of the locations of the steam line outputs and the fact that the steam rises to the top of the girders under the tarps. Recorded differences in the concrete temperatures for this study vary by as much as 48º F.

4. The control cylinder displays early age shrinkage, possibly due to the autogenous shrinkage, while the girders display early age expansion, possibly due to thermal effects.

5. The strain profile of the girders during steam curing when high temperature changes occur is significantly influenced by the assumptions used in the temperature correction for the vibrating wire gages. The presence of water in the concrete at early ages may contribute to the inability to know the coefficient of thermal expansion of the concrete at early ages with good certainty.

6. The presence of locked in mechanical strains due to thermal gradients has been suggested as a possible cause of additional prestress loss during the casting phase. However, the thermal gradients in the girders for this study are minimal for approximately five hours after curing. It is believed that the concrete has initially gained sufficient strength during this time to cause the strand and the concrete to act compositely, preventing locked in mechanical strains due to the thermal gradients from occurring.

7. Thermal gradients that exist at detensioning may influence measured curvatures at detensioning. But, for this study, the thermal gradients at detensioning predict a change in curvature that is opposite to what is observed.

8. Measured changes in strain (and curvature) at detensioning indicate that the loss of prestress force during casting may be less than predicted by others. Measuring the change of stress in the strands outside of the girder may not be a good indicator of actual stress loss in the strand at detensioning. However, without monitoring the changes in strain at detensioning both within the girders and in the strands outside the girders, it is impossible to know the exact prestress loss at detensioning.
8.4 Static Phase Conclusions

1. Immediately after casting the deck on a girder in a continuous system, some positive restraint moments begin to develop due to thermal expansion of the deck. The positive restraint moments continue to develop for approximately one day.

2. After an age of one day, negative restraint moments begin to develop, and continue to develop rapidly until the deck is approximately four days old. After a deck age of approximately four days, some negative restraint moments continue to develop, but at slower rate.

3. The magnitudes of the restraint moments observed are less than predicted by conventional analysis. At early deck ages (less than seven days) conventional analysis can not be used to predict the restraint moments. At the later deck ages observed, one or more mechanisms prevent conventional analysis from predicting the correct magnitudes of the restraint moments.

4. Significant compressive strains are measured in the reinforcing in the deck throughout the static phase, indicating that shrinkage of the deck is occurring. However, the shrinkage which occurs does not completely manifest itself as a differential shrinkage causing changes in restraint moments as measured by the changes in reactions.

5. Possible reasons for the difference in the restraint moments observed and the restraint moments predicted include: restraint due to the reinforcing in the deck, softening in the continuity diaphragm connection, shrinkage of deck and girder occur at approximately the same rate, the deck shrinkage at the top of the deck is greater than at the bottom of the deck, and the extensibility of the deck concrete reduces the forces to the top of the girder.

8.5 Service and Cyclic Phase Conclusions

1. The predicted initial positive cracking moment capacity at the interface of the end of the girder and the diaphragm is lower than current recommendations predict. The fact that the end of the girder acts as a cold joint causes debonding at the section, reducing the cracking moment capacity.

2. Both continuity tests, Test 1 with the extended strands and Test 2 with the extended 180
degree U bars, display acceptable behavior during the initial service tests. Test 2 is slightly stiffer, with average crack openings at the bottom of the interface of 0.022 in. versus 0.029 in. for Test 1 at a loading of 1.2Mcr. Therefore, the crack openings of Test 1 are 1.3 times larger than the crack openings of Test 2 following the service tests.

3. Following the cyclic phase testing, both test sections are capable of withstanding anticipated service load moments (up to 1.2Mcr). But, during the cyclic phase of testing, Test 1 begins to display some undesirable behavior. The crack openings begin to increase at approximately 5,000 thermal cycles and continue to increase until at the end of testing. At the end of testing, crack openings for Test 1 are 2.6 times larger than crack openings for Test 2, 0.047 in. versus 0.018 in. respectively at a loading of 1.0Mcr.

4. Both Test 1 and Test 2 display good repeatability during the negative moment testing phase.

5. Both Test 1 and Test 2 exceed the predicted ultimate positive moment capacities by ratios of 1.32 and 1.35 respectively. The predicted ultimate positive moment strength for Test 1 is based on the method proposed in “End Connections of Pretensioned I-Beam Bridges” (Salmons 1975). The predicted ultimate positive moment strength for Test 2 is based on current design procedures using Whitney’s Equivalent Stress Block (ACI Building 2002).

6. For Test 2, minor cracking in the ends of the girder was noticed at all four corners of the connection.

7. For Test 3, which did not have a continuity connection, two distinct cracks instead of one developed during the service load testing.

8. For Test 3, after the initial deck damage which occurs during the initial service load testing, not much additional damage occurs during the cyclic testing.

### 8.6 General Recommendations

1. For design, thermal restraint moments determined from the “AASHTO Guide Specifications – Thermal Effects in Concrete Bridge Superstructures, 1989”, should be considered. For ages of continuity over approximately 60 days, the thermal restraint
moments generally control over restraint moments due to creep and shrinkage. From this study, it is possible that final restraint moments due to both thermal effects and creep and shrinkage effects may be less than predicted. However, the restraint moments due to thermal influences measured during the study appear equally significant, or even more significant, than the restraint moments due to creep and shrinkage.

2. Since the temperature correction for vibrating wire gages is sensitive to the actual coefficient of thermal expansion of the concrete, any experimental testing using vibrating wire gages should attempt to monitor the actual coefficient of thermal expansion of the concrete during the testing. This is especially important at early ages, when the influence of water in the system may significantly alter the coefficient of thermal expansion of the concrete.

3. During fabrication of girders, the fabricator should consider the potential prestress losses in the strands due to heating or cooling of the strands in the forms prior to casting the concrete by either slightly altering the jacking force or controlling the strand and concrete temperatures. The fabricator should also monitor bed temperatures near the top of the forms, where the steam tends to collect and the highest temperatures are present. Any attempt to measure the actual force in the prestressing strands should include both the monitoring of the strains in the girder and the strains in the strands outside the girder at detensioning. Once the gains sufficient strength, the change in force of both at detensioning are needed to estimate the effective prestress force.

4. Conventional analysis can not be used to estimate restraint moments at early ages of the deck. A modified approach is presented which predicts restraint moments at early ages of the deck (less than 7 days), but overestimates the magnitude of the restraint moments at later ages. Additional research is needed to identify the exact mechanisms that take place during differential shrinkage of composite systems.

5. To determine the moment that will cause initial cracking, the initial positive cracking moment capacity of the girder at the diaphragm should be determined with an allowable tensile capacity of the concrete, $f_t$, equal to $(1/4)(w f'c)^{0.5}$ where $w$ is the unit weight of the concrete (in pcf) and $f'c$ is the compressive strength of the concrete (in psi).
6. Although the continuity connections for both Test 1 and Test 2 perform adequately under service and ultimate loads, the connection for Test 2, with the 180 degree U bars, is recommended for use. A detail is provided in Appendix I. The connection remains stiffer during the testing and is expected to provide for a better long term connection. The cyclic testing performed on Test 2 induced stresses of 31.6 ksi tension in the bars. The maximum service stresses induced at 1.0Mcr and 1.2Mcr were 37.5 ksi and 45.0 ksi in tension, respectively. Two transverse joints above the ends of the girders instead of one should be used if the continuous deck detail tested in Test 3 is implemented.

8.7 Recommended Design Procedure

The following procedure is recommended for design of PCBT girders made continuous:

1. It is recommended that PCBT girders that are to have a continuous deck be detailed to have a continuity diaphragm. The continuity diaphragm benefits the structure by adding redundancy and reducing the magnitudes of crack widths on the deck (when compared to a structure made continuous without a continuity diaphragm). For negative moment continuity, the reinforcing steel in the deck should be designed to resist the tensile loads using factored loading.

2. The ends of the girders should be detailed to remain flush with the continuity diaphragm. Embedment of the girders into the diaphragm is not recommended.

3. For positive moment, the continuity diaphragm should be designed to resist the maximum factored anticipated service load. This load should be determined as the maximum of the restraint moment due to thermal gradient or the restraint moment due to creep and shrinkage effects. For ages of continuity over 60 days, the thermal restraint moments will usually dominate. Ultimate strength analysis (for the extended bars) or the method proposed by Salmons shown in Section 2.3 (for the extended strands) should be used to determine the capacity of the section. A load factor of 1.0 is recommended for factoring the service load. It is believed that the prediction of the restraint moments using a conventional analysis sufficiently over-predicts the magnitude of the moment at the continuity connection, allowing for the load factor of 1.0.

4. In lieu of an advanced analysis considering both early and late ages of continuity, thermal
effects and shrinkage and creep effects should not be considered for the design of the girders or the negative moment connection of the diaphragm. For service conditions, the girders should be designed as simple spans for dead and live loads. For ultimate strength conditions, the girders should be designed using Load Factor Design assuming a fully continuous system. The negative moment above the continuity connection should also be designed based on a fully continuous structure using Load Factor Design, recognizing that the additional capacity in negative moment will help to redistribute any restraint moments due to creep, shrinkage, or thermal effects that would otherwise increase moments at midspan. Analytical work in the NCHRP 322 Report (with confirmation by NCHRP 519 Report and this project) indicates that a structure with a continuity diaphragm is able to transfer moments through the joint during the service load range. A loss in continuity is not anticipated until loading approaches failure. Even though during the service load range the connection is able to transfer moments, additional rotations are required to transfer the moments because of cracking in the diaphragm. The benefits of reduced moments in mid-span due to the continuous design may not be realized fully due to the fact that the structure may have to rotate an additional amount to allow the reduced loading to occur. The uncertainty caused by this fact, coupled with the fact that the restraint moments induced because of the continuity can increase the moments at mid-span, lead to this recommendation.

5. Whether or not cracking due to positive restraint moments will occur at a continuity connection can be checked by calculating the cracking moment capacity based on an upper bound for the tensile capacity at the bottom of the interface of \((1/4)(w't)\)\(^0.5\). Gross section properties at the interface using the material properties of the continuity diaphragm should be used in determining the actual cracking moment capacity. The full slab width and profile of the girder should be used in determine the gross properties.

6. It should be recognized that the interface of the girder and the continuity diaphragm will most likely crack or debond at a fairly low load level (lower than the conventional calculated cracking moment, \(M_{cr}\)). The edges at the end of the girder diaphragm interface should be sealed with a material able to withstand movements (up to 0.050 in.) at the time of initial construction. This type of material may require a backer rod.
7. For constructability, the design service moments (due to either thermal effects or creep and shrinkage effects) at the diaphragm should be kept below the conventional 1.2Mcr. This limit is based on the amount of steel that can easily extend from the bottom of a girder for the positive moment connection and the fact that no tests have been performed on girders with larger amounts of steel.
References


5. American Concrete Institute (ACI), (2002), *Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (ACI 318R-02)*, Reported by ACI Committee 318, Farmington Hills, MI, 48333-9094.


34. McDonagh, M., (2001), *RMCalc*, computer software, member Washington State Department of Transportation’s Alternate Route Project.


Appendix A: Development of an Optimized Continuity Diaphragm for New PCBT Girders
VIRGINIA COOPERATIVE CENTER FOR
BRIDGE ENGINEERING

RESEARCH PROJECT
FY 2002

Development of an Optimized Continuity Diaphragm
For New PCBT Girders

By
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Tommy Cousins
Virginia Tech

Jose Gomez
Virginia Transportation Research Council

December 1, 2001
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RESEARCH SIGNIFICANCE

With the implementation in Virginia of the new Prestressed Concrete Bulb-Tee (PCBT) girders, the need arises for a well-detailed continuity diaphragm. Simple span girders made to be continuous for live load and superimposed dead loads are more efficient and inherently safer than ordinary simple span girders. Also, by casting the deck continuously, high maintenance joints are eliminated over interior supports and rider comfort is greatly improved. The elimination of joints not only enhances the durability of the deck, but also prevents premature deterioration of other super- and sub-structure components, which can be caused by water and deicing salts leaking through faulty joints.

The continuity diaphragm, however, must be properly analyzed, designed, detailed and constructed. Improperly detailed or constructed continuity diaphragms can develop excessive cracking with time and themselves become a high maintenance element of the bridge. Also, the degree of cracking in the diaphragm will affect the degree of continuity actually achieved by the adjacent spans, and hence the magnitude of the mid-span live load moments.

The objective of this proposed research is to develop and test several continuity diaphragm details for PCBT girder bridges, and propose an optimum continuity detail to VDOT.

WORK PLAN

Introduction
Simple span precast concrete girders can be made continuous by the casting of a continuous deck slab and a continuity diaphragm over the interior supports. In this way, the bridge girders initially carry their own dead loads as simple span girders, but carry live loads and super-imposed dead loads as continuous composite girders. Whether the deck is carried as simple span or continuous depends on the casting sequence and reinforcing details.

The diaphragm must be designed for both the negative moments due to super-imposed dead load, live load and differential slab shrinkage, and the positive moments due to positive thermal gradients and the moments that develop over time due to the restraint of creep induced end rotations. Many previous researchers have endeavored to analyze the diaphragm and propose continuity diaphragm details (1,2,3,4,5). The currently unresolved issues remain:

1. How much, if any, positive moment reinforcement should be provided in the continuity diaphragm?
2. If positive moment reinforcing is provided, how much restraint moment develops?
3. How effective is the diaphragm in maintaining continuity between the spans?
The currently underway NCHRP Project 12-53 is examining these issues, and developing and testing details for standard AASHTO girder shapes. The project proposed herein will build on the results of the NCHRP project, currently projected to be completed in the summer of 2002, and develop details for the new PCBT shapes. In addition, this project will consider the effects of thermal gradients, which were not within the scope of the NCHRP project. With the goal of recommending to VDOT an optimum detail, the proposed research will involve the following tasks:

Task 1. Literature Review and DOT Survey
The above-mentioned NCHRP project has completed an extensive literature review, some of which has been recently reported (6). In addition to the review of these publications, state DOT's will be surveyed to determine current analysis practices and standard details for continuity diaphragms. Particular attention will be paid to states using bulb-tee shapes.

Task 2. Analytical Studies
Analytical studies will be performed utilizing the methods outlined by Mirmiran et al.(6). The authors present a flexibility based analytical method which considers the non-linear moment curvature relationships for the different bridge components (girders and diaphragms). The time dependent effects of creep and shrinkage of the girders, differential shrinkage of the deck with respect to the girders, and prestress losses caused by creep, shrinkage and relaxation are all taken into account. The modeling method predicts the range of positive and negative moments expected in the diaphragm and the influence of the reinforcement percentage in the diaphragm on the magnitude of the restraint moments. For this project, the influence of positive and negative thermal gradients will also be considered.

Parametric studies will be performed to investigate the full range of PCBT girder sizes, several span lengths and span arrangements for each girder size, various reinforcement ratios in the diaphragm and the age of the girders at the time of creation of continuity. These parametric studies will indicate the optimum reinforcement ratios for the diaphragms, and the range of positive and negative moments the girders and diaphragms will experience in service.

Task 3. Detail Development
Based on the parametric studies, the outcome of the NCHRP project, review of existing details and discussions with VDOT design engineers, a variety of candidate diaphragm details will be developed and three will be selected for testing. In addition to the input of the VDOT designers, opinions on the constructability of the details will be sought from bridge contractors and precasters.

Task 4. Detail Testing
The three details selected for testing will be those considered to have the highest likelihood of optimum performance, and may utilize a variety of methods for creating continuity, such as:

- Reinfreching steel extending from the girders into the diaphragm
- Prestressing strand extending from the girders and bent upwards into the diaphragm,
- Poor-boy continuity in which only the deck slab is continuous over the pier
- Post-tensioning across the diaphragm.

Short PCBT sections will be cast at a precast plant. At the plant, any required embedded instrumentation will be installed. Then, the beams will be transported to the structures lab. For each of the three prototype diaphragms, the diaphragm will be cast between beam-ends, a composite deck will be placed, and the system will be tested (see Figure 1.). The tests will be designed to examine many aspects of diaphragm performance including:
  - Precracking behavior,
  - Positive cracking moment,
  - Post-cracking behavior,
  - Behavior under cyclic loads,
  - Negative cracking moment,
  - Ultimate negative moment capacity.

As shown in Figure 1., the center support of the test arrangement is to be a hydraulic ram. This ram will be locked off during construction. After the deck and diaphragm have been cast, additional dead load blocks will be placed to create a reaction at the ram similar to that calculated for the actual bridge interior supports.

The first phase of each test will involve lowering the ram to reduce the reaction and induce positive moments in the diaphragm. As the load is decreased incrementally, deflections and strains will be recorded. The first cracking load will be noted and cracks marked. Subsequently, the load will be decreased further to mimic the calculated maximum positive moment expected in the actual structure. The diaphragm response will be monitored by measuring internal and external strains, recording crack patterns and crack widths, and measuring changes in deflections at the ram and along the girders.

During the second phase of each test, the ram will be raised to study the response of the diaphragm to negative moments. The change in response of the diaphragm before and after the closing of the positive moment cracks will be of particular interest. The load will be increased incrementally and the onset of negative moment cracking will be recorded.

The third phase of testing will involve cycling the load between the worst expected service negative and positive moments. As the cyclic load is applied, changes in diaphragm response will be investigated. Following cyclic testing, the load will be increased to cause failure due to negative moment.

**Task 5. Data Analysis**
Following the completion of all physical tests, the data will be analyzed to determine the optimum detail. Then, the detail for the small test bulb-tees will be modified for the other bulb-tee girder sizes.
REPORTS

Following the analysis of the test data, recommendations will be made to VDOT for an optimum diaphragm detail, including construction scheduling. Interim reports will be submitted in November and May of each year of the project.

TIME REQUIREMENTS

Project time period is 24 months including the preparation of the final project report. The following table presents the time requirements for each project task and the preparation of the final report. The project period is July 1, 2002 to June 30, 2004.

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COOPERATIVE FEATURES

The proposed project is a cooperative study combining the resources of researchers and designers from Virginia Tech (Carin Roberts-Wollmann, Tommy Cousins and graduate student), VTRC (Jose Gomez) and VDOT (John Martiu and S.L. Hite). It is expected that the cooperative effort will involve the evaluation of existing details, NCHRP test results and analytical studies from this project and the development of the most promising diaphragm details for laboratory investigation.

REFERENCES

Appendix B: PCBT Girder Information
Maximum number of strands in Row number
1: 14
2: 14
3: 12
4: 6
5 & higher: 2

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Notes to designer:
1. Strands: Seven-wire Grade 270 low-relaxation strands, 2" grid (spacing), drape points at 0.4L and 0.6L.
2. Slab thickness: 7 1/2" incl. 1/2" w.s.
3. Dead Loads: Non-Composite = 0.20 kips/ft. excl. weight of beam and slab.
    Composite = 0.27 kips/ft.
4. Live Load: HS20-44.
5. Allowable tension = 6-\text{ft} lb.
6. Strand designs shown with asterisks(*) exceed max. number of draped strands(14) or max. total uplift force(40 kips) for strand restraining devices. Additional investigation by designer is required.
Appendix C: Influences of Design Methods and Assumptions on Continuity Moments in Multi-Girder Bridges
INFLUENCES OF DESIGN METHODS AND ASSUMPTIONS ON CONTINUITY MOMENTS IN MULTI-GIRDER BRIDGES

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ABSTRACT

Over the past fifty years, many states have recognized the benefits of making precast, prestressed multi-girder bridges continuous by connecting the girders with a continuity diaphragm. Although there is widespread agreement on the benefits of continuous construction, there has not been as much agreement on either the methods used for design of these systems or the details used for the continuity connections.

Recently, a survey performed in the NCHRP 12-53 project confirmed that differences still exist in the design of restraint moments and the detailing used to resist this type of moment.1

To aid designers in choosing the most appropriate method, an analytic study was undertaken to compare the differences in the predicted continuity moments for different design methods and assumptions over a range of commonly used systems of precast, prestressed PCBT girders and cast-in-place slabs. It was found that the predicted positive thermal restraint moments could be significant, similar in magnitude to the actual positive cracking moment capacities. This paper presents these results along with the results of a study performed to determine how significant the differences are between four different methods used to predict restraint moments due to creep and shrinkage.

Keywords: Continuity, Restraint, Creep, Shrinkage, Prestress Losses, PCBT Girders
INTRODUCTION

It was recognized as early as the late 1950’s in the United States that making a simple span I-girder bridge continuous by connecting the ends of the girders with a continuity connection provided several benefits. The continuity helps to reduce deflections and moments at midspan, eliminates the joints and as a result reduces deterioration of the concrete from deicing salts, and provides for some reserve load capacity. Today, because of the consideration of the influence of restraint moments at midspan, there is some debate as to whether midspan moments are actually reduced. However, the remaining benefits are commonly recognized and many states are requiring multi-span I-girder bridges to be designed and detailed as continuous.

Although there is widespread agreement on the benefits of continuous construction, there is not as much agreement on either the methods used for design of these systems or the details used for the continuity connections. Many methods have been proposed for determining the moments at the continuity connections. A recent survey performed as part of the NCHRP 12-53 project revealed that about 35 percent of those responding use the PCA Method while about 9 percent use the NCHRP 322 method to predict the continuity moments. In addition, the survey revealed that about 48 percent of those responding use some type of standard detail for the positive moment connection while about 25 percent do not use a positive moment connection. Therefore, the three most commonly used methods for determining the continuity moments are: (1) the PCA method, (2) the method outlined in the NCHRP 322 document, and (3) providing a standard detail (perform no calculations). When the PCA Method and the NCHRP 322 Method are used to predict the restraint moments of the same system, the predicted moments can often differ significantly for girder and slab systems commonly used in highway construction. Also, thermal restraint moments are not considered in any of three most commonly used methods.

MOTIVATIONS

Since there is not good agreement on the best way to design for continuity moments in multi-girder bridges, designers are faced with a difficult choice as to which method to use, or to assume it acceptable to forego the calculations and simply provide a standard detail. Also, designers who work in multiple states may have to provide details that they are not familiar with in order to meet the design criteria for a given state.

To aid designers in choosing the most appropriate method, an analytic study was undertaken to compare the differences in the predicted continuity moments for different design methods and assumptions over a range of commonly used systems of Precast Concrete Bulb-Tee (PCBT) girders and cast-in-place slabs. In addition to the PCA and NCHRP-322 Methods, two other methods were developed by the authors and the effects of thermal gradients were also investigated. Although there is disagreement on the prediction of both positive and negative restraint moments, the prediction of the positive restraint moments causes designers the most problems. Because the positive restraint moments cause tension in the bottom of
the girder at the ends, provisions must be made to resist the tension. To resist this tension, extended strands, extended mild steel, or some other reinforcement extending from the ends of the girders is required. This paper presents the results from this study.

BACKGROUND OF CONTINUITY DESIGN

One of the first major studies performed to address the issues of prestress girders with a cast-in-place deck made continuous was performed in the Research and Development Laboratories of the Portland Cement Association and was published in a series of Development Department Bulletins beginning in 1960. Bulletin D34 summarized the initial pilot tests which were designed to investigate the strength of the continuity connection in negative bending as well as the strength and moment redistribution of the continuous girders with a negative moment connection. Another part of the studies undertaken by the Portland Cement Association was an investigation into the effects of creep and shrinkage on continuous bridges. This study, which monitored two half-scale two span continuous girders over a period of approximately two years, was presented in Development Department Bulletin D46. The two girder systems investigated were similar except that girder system 1/2 had no positive moment connection while girder system 3/4 had a positive moment connection made of 4-No. 3 hook bars projecting from the ends of each girder. To account for the effects of scaling, large dead load blocks were hung from the structure throughout the testing. Restraint moments were determined by monitoring the change in reactions of the supports during the time period. It was concluded that both the Rate of Creep Method and the Effective Modulus Method could be used for prediction of the positive moments induced by the combined effects of creep and shrinkage. Since there is a shortage of long term studies of restraint moments caused by creep and shrinkage, the results of these two tests have been used as the basis to compare many other design methods that predict creep and shrinkage restraint moments.

In 1969, the Portland Cement Association released an engineering bulletin which was based primarily on the earlier research documented in the Development Department Bulletins released in the early 1960’s. This engineering bulletin, the “Design of Continuous Highway Bridges with Precast, Prestressed Concrete Girders”, became the standard for continuous design, and is still used by a considerable number of designers in the year 2004, thirty-five years later. The design guide contains two parts, the first part gives guidance on the procedures necessary in continuous design and the second part contains a design example.

From 1971 to 1974, the Missouri State Highway Commission in cooperation with the Federal Highway Administration investigated the use of extended prestressing strands to develop the positive restraint moment in a continuity connection for prestressed girders. The results of this research were published in a report titled “End Connections of Pretensioned I-Beam Bridges”. The research contained in the report was divided into two main topics. First, a study was performed to determine the relationship of the embedment length of untensioned prestressing strands to the strength of the strands. The second study focused on the
embedment length and strength relationship of the strands in full scale I-beam continuity connections.

In the mid 1980’s, the National Cooperative Highway Research Program (NCHRP) undertook a project designed to address the behavior and design methods of precast, prestressed girders made continuous. This study was performed primarily at the Construction Technology Laboratories in Skokie, Illinois, and was released in 1989 as NCHRP Report 322, “Design of Precast Prestressed Girders Made Continuous”. This report investigated many of the assumptions and design requirements which were presented in the earlier 1969 PCA report. In an attempt to determine the current state of practice throughout the country, the project’s First Task included a literature review and a questionnaire. Also included in the First Task was a limited number of tests performed on steam cured concrete specimens to investigate creep and shrinkage specifically for steam cured precast, prestressed concrete with loadings at early age (two day release of strands). The Second Task included parametric studies performed using some existing computer programs and two newly created programs to determine the influence that certain material properties and design assumptions would have on the resulting service moments. The Third Task included analytic procedures to investigate the flexural strength of the members along their lengths. The final task, the Fourth Task, concentrated on determining the strength and service requirements for the continuity connections over the interior supports.

Results of the questionnaire indicated that of the 42 respondents who provide positive moment reinforcement in the continuity connection, 30, or approximately 71 percent, used the PCA method as the primary method for design. Concerning the type of reinforcing used for the positive moment connection, 18 respondents used embedded bent bars and 21 respondents used extended strands, a 46 to 54 percent split.

Considering the positive moment which may develop at an interior support, it was found that “...providing positive moment reinforcement has no benefit for flexural behavior of this type of bridge...” and that “...the provision of positive moment reinforcement at the supports is not recommended”. Although it was indicated that the positive moment reinforcing over a support can help to reduce the size of the cracks that may develop, the presence of the positive moment reinforcing was found to have a negligible effect on the positive moment at the midspan of the structure. Although providing positive moment reinforcing at an interior support would provide some benefit to the structure by introducing continuity for superimposed loads, this benefit was found to be offset by two factors. First, positive moment caused by the positive restraint moment above the support causes an increase in the positive midspan moment. Second, as a positive moment develops over a support, cracking occurs in the bottom of the continuity region. The superimposed dead and live loads added to the structure then have to cause sufficient rotation in the section to overcome the cracking before the benefits of continuity could be realized. In other words, due to the cracking in the bottom of the continuity connection, full continuity could not be established. Therefore, it was recommended that positive moment reinforcing not be used. Responses from the questionnaire indicated that some states had already used continuity connections with no positive moment reinforcing. Responses by California, Florida, Minnesota and Wisconsin all
indicated that they had experience using continuous decks with no positive moment reinforcing and that they had not experienced any problems with this type of detail.

The project NCHRP 12-53, which has recently been completed, was a joint venture between the University of Cincinnati and Ralph Whitehead Associates. Similar to the NCHRP 322 project, this project first performed a survey to determine the state-of-the-practice for precast, prestressed girders made continuous. The results from this survey can be found in the article by Hastak et al. Next, the project investigated the different prediction models and developed a new model to predict the magnitude of continuity moments. Six full scale stub specimens were fabricated and tested to determine the effectiveness of different continuity details. Two full scale and full length specimens were also fabricated and tested. Based on the results of the survey and the testing, recommendations were made for design and detailing of continuity connections.

It was concluded that restraint moments caused by temperature effects were significant. As stated in the report, these restraint moments, which are usually not considered in design, can be as significant as restraint moments due to live loads. It was also recommended that the amount of positive moment steel in the continuity connection not be greater than that which will provide a moment capacity of 1.2 times the cracking moment. If higher restraint moment capacities are required, a minimum age of continuity should be required in the contract documents to prevent such a high moment from developing.

Parametric studies predicted that for the positive moment steel in a continuity connection, as the amount of steel increases, the continuity of the system increases, causing higher negative moments over the supports and lower positive moments at midspan. Also, as the amount increases, the cracking in the bottom of the continuity connection decreases while the positive restraint moment increases. Although the parametric studies indicated that cracking in the continuity diaphragms reduces the continuity of the system, this reduction was not observed in the full size test specimens except at near ultimate loads. When cracking at the bottom of the connection first occurred, the crack did not immediately extend all the way up into the slab as predicted by the parametric studies.

METHODS CONSIDERED

OUTLINE OF METHODS

PCA Method - Four different methods used to predict the restraint moments due to creep and shrinkage were considered. The first method, the PCA Method, was taken directly from the 1969 Engineering Bulletin without any modifications. Although the method indicates that laboratory data may be used to predict creep effects, many designers use the tables provided in the method for prediction of creep and shrinkage. The method assumes that the proportion of ultimate creep or shrinkage that occurs over time can be estimated using a figure provided and that the ultimate creep or shrinkage of both the girder and the deck can use the same figure. Also, it is assumed that the ultimate shrinkage of the deck and the girder will be the
same, 0.600x10^{-3} at 50 percent relative humidity. Final restraint moments are determined by multiplying the instantaneous restraint moments by the time dependent factors. For loads applied instantaneously, such as the initial prestressing force and the dead loads, the moments are multiplied by (1-e^{-ρ}) and for moments applied slowly over time, such as the shrinkage restraint moments, the moments are multiplied by (1-e^{-ρ})/ρ, where ρ is the creep coefficient.

RMCalc Method - The second method considered is called the RMCalc Method. RMCalc version 2.1.0 is an algorithm copyrighted by Michael McDonagh of Entranco, Inc. The algorithm is available online as part of the Washington State Department of Transportation’s Alternate Route Project, where terms and conditions of its use are made available. It uses the same design procedure as does BRIDGERM which was developed as part of the NCHRP 322 project. The main difference is that RMCalc is written in Visual Basic while BRIDGERM was written in FORTRAN. Sample calculations that were performed in both programs showed that the two gave the same results.

The program determines restraint moments in a continuous girder system due to creep and shrinkage. To determine the restraint moments, an incremental time step solution is performed. The program uses ACI-209 creep and shrinkage models published in 1982. Prestress losses are determined based on the PCI Committee on Prestress Losses recommendations published in 1975. The influence of the reinforcing in the deck on the shrinkage of the deck is also considered. Unlike some programs and design procedures, RMCalc considers the actual length of the continuity diaphragm in the direction of the span as a small interior span. Special routines are used to determine if the restraint moments which are created ever produce a situation where the reaction becomes upward (or negative) at the end of girder and diaphragm interface.

The program requires the input of the ultimate creep coefficient and shrinkage for the girder and the ultimate shrinkage of the deck. To determine these values, the other three methods were first computed and the average values from these three methods were used for the input.

Comparison Method 1 - The third method considered, which will be called Comparison Method 1, was developed by the authors and is based on some of the procedures of the PCA method with modifications. Current provisions of the American Concrete Institute (ACI) committee 209 were used to model the creep, shrinkage, and age-adjusted effective modulus over time. The concrete for the girder and the deck were considered separately, and different ultimate creep and shrinkage values were obtained for each. Final restraint moments are determined by multiplying the instantaneous restraint moments by the time dependent factors which include the influence of concrete ageing, considered with the ageing coefficient X. For loads applied instantaneously, such as the initial prestressing force and the dead loads, the moments are multiplied by the quantity ρ/(1+X ρ). For moments applied slowly over time, such as the shrinkage restraint moments and prestress losses, the moments are multiplied by 1/(1+X ρ), where ρ is the creep coefficient.
Comparison Method 2 - The fourth method considered, which will be called Comparison Method 2, was also developed by the authors and is based on The CEB-FIB, Model Code for Concrete Structures, 1990 (MC-1990), which provides predictions for the time effects of temperature, shrinkage, and creep on concrete. This model code, which was developed in Europe, is based on SI units and will be presented in these units with conversions to English units where appropriate. The code is intended for concretes having characteristic compressive strengths, $f_{ck}$, which range from 12 Megapascals (MPa) to 80 MPa (1.74 ksi to 11.6 ksi). The characteristic compressive strength is the compressive strength of a cylinder, which is often referred to as the specified compressive strength, $f'_{c}$ in the United States. Instead of the characteristic compressive strength, the model uses the mean compressive strength, $f_{cm}$, which can be taken as:

$$f_{cm} = f_{ck} + 8 \text{ MPa}$$

where 8 MPa is equal to 1.16 ksi. The model is intended for sustained loads that produce stresses which are less than forty percent of the mean compressive strength, relative humidity from 40 percent to 100 percent, and temperatures from 5°C to 30°C (41°F to 86°F). This method is based on a design example presented in a book by Ghali and Favre where a flexibility based approach is used for the distribution of moments. The change in rotation over a restrained joint, $\Delta D$, is first determined with the restraint removed. If the load is slowly applied, then the change in rotation is determined using the age adjusted effective modulus of elasticity, $E_{adj}$, which is given by:

$$E_{adj} = \frac{E}{1 + X\phi}$$

where $E$ is the modulus of elasticity at 28 days, $X$ is the ageing coefficient, and $\phi$ is the creep coefficient. An age adjusted flexibility coefficient, $f$, is then determined for all loads:

$$f = \frac{l}{E_{adj}I} \left( \frac{1}{a} + \frac{1}{b} \right)$$

where $l$ is the span length, $I$ is the moment of inertia of the section, and $a$ and $b$ are coefficients depending on the geometry of the continuous system. If the joint adjacent to the joint under consideration is an exterior joint, the values for $a$ and $b$ are 3, otherwise the values are 2. Finally, the ultimate restraint moment, $\Delta F$, is determined by:
Thermal Gradients - Although most currently used design methods for continuous girders in multi-girder bridges do not consider thermal effects, it has been found that thermal effects can have a significant influence on the design process. In fact, recent full scale experiments performed for the NCHRP 12-53 project have indicated that the thermal effects can be as significant as the live load effects. Results of this study have indicated that the daily temperature changes can cause end reactions to vary by as much as 20 percent per day. Taking these changes in end reactions and multiplying by the span length provides a significant restraint moment due to the temperature change. Therefore, the restraint moments due to positive and negative thermal gradients have been calculated and the results of the positive gradients are presented.

The commentary to the LRFD Specifications states that “...open girder construction and multiple steel box girders have traditionally, but perhaps not necessarily correctly, been designed without consideration of temperature gradient...” To consider the effects of temperature gradient, guidelines are provided in the “AASHTO Guide Specifications – Thermal Effects in Concrete Bridge Superstructures – 1989” These guidelines divide the United States into four Maximum Solar Radiation Zones. Zone 3 was used for this study. The temperature differentials for a concrete superstructure are provided for both a positive and a negative temperature gradient. The positive thermal gradient used is shown in the Design Example at the end of the article.

In order to determine the effects of the temperature gradient, the structure must first be made determinate by removing a sufficient number of internal redundancies. After the internal redundancies are removed, the self-equilibrating stresses (or forces) are determined. The redundancies are then reapplied, producing the continuity stresses (or forces).

First, the stresses created due to the temperature gradient are calculated assuming that the structure is totally restrained. These longitudinal stresses, $\sigma_t(Y)$, are determined at a distance $Y$ from the center of gravity and are given as:

$$ \sigma_t(Y) = E\alpha T(Y) $$

where $E$ is the modulus of elasticity, $\alpha$ is the coefficient of thermal expansion, and $T(Y)$ is the temperature at the given distance $Y$ from the center of gravity of the system. Next, the restraining axial force, $P$, is determined by integrating over the depth of the structure:

$$ P = \int_Y \sigma_t(Y)b(Y)dY $$

where $b(Y)$ is the section width at location $Y$. Likewise, the restraining moment, $M$, is determined by integrating the product of the stress, the width, and the distance from the centroid over the height of the structure. It can be determined by:
\[ M = \int \sigma_t(Y)b(Y)YdY \]

The self equilibrating stresses, \( \sigma(Y) \), are then determined by:

\[ \sigma(Y) = \sigma_t(Y) - \frac{P}{A} \frac{MY}{I} \]

where \( A \) is the area of the section and \( I \) is the moment of inertia of the section. Any redundancies that were removed to make the structure determinate are then reapplied. The self-equilibrating stresses, \( \sigma(Y) \), or self-equilibrating forces (P and M), are then redistributed to produce the continuity stresses or forces.

**Time Functions** - A summary of the different time functions used to determine the final restraint moments is shown in equation 9. For the PCA Method, the effect of prestress loss is not determined directly. Instead, the effective prestress force is used in determining the effect of the prestress force. For the RMCalc Method, a time step procedure is used where the creep coefficient over the time step, \( \phi_t \), is determined as the difference of the creep coefficient at the end of the time step and the creep coefficient at the beginning of the time step. Prestress losses are indirectly accounted for by determining creep and shrinkage in the concrete using the 1982 ACI 209 and prestress losses using the 1975 PCI.\(^3\) Method 1 uses the aging coefficient \( X \) while Method 2 first determines the rotation at the continuous joint \( \Delta \), by using either the 28 day modulus of elasticity or an age adjusted modulus and then divides this rotation by an age adjusted flexibility factor, \( f \), to determine the final moments.

**CASES CONSIDERED**
For the analytical study, the predicted restraint moments due to creep and shrinkage of the concrete for the four cases above and the thermal gradients were calculated for 12 combinations of girder size, slab size, span and number of prestressing strands which came from preliminary design tables. An outline of the combinations is shown in Table 1.

Table 1 – Outline of Combinations Considered

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<th>Girder Mark</th>
<th>Span Length, ft</th>
<th>Number of Strands</th>
<th>Slab Width, ft</th>
<th>Identification</th>
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<td>130L/46S/10D</td>
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The depth of a girder is given by the last two digits in the mark, a PCBT 45 girder is 45 in. deep. For the prestressing strands, the strands in the web were draped with a harping point assumed to occur at 0.4 times the span length. The strands used were seven wire ½ in. diameter, low relaxation Grade 270 with an assumed release stress of 0.75 times $f_{pu}$ and an assumed stress at service of 0.6 times $f_{pu}$. For the beam spacing of 6 ft 0 in., a deck thickness of 7 ½ in. including a ½ in. wearing surface was assumed and the non-composite and composite loads were assumed to be 0.20 and 0.135 kips/ft respectively. For the beam spacing of 10 ft 0 in., a deck thickness of 8 ½ in. including a ½ in. wearing surface was assumed and non-composite and composite loads were assumed to be 0.28 and 0.16 kips/ft respectively. For all cases, a 1 ½ in. bolster was assumed, except for the thermal gradient calculations.

For the PCA method, the final prestressing force with losses was used for the calculations. For the other three methods, the percentage of prestress losses which have occurred up to the time of continuity was determined using the provisions of the AASHTO Standard Specifications and the actual prestressing force at the time of continuity was adjusted to account for prestress losses which had occurred up to that point.¹⁵
Restraint moments were calculated for seven different girder ages at the time when the continuity was established. The deck and the continuity connection were assumed to occur at the same girder age. Girder ages of 14, 28, 60, 90, 120, 180, and 365 days were analyzed. Theses ages were chosen to show how the predicted ultimate restraint moment will vary due to different assumed ages of continuity. To be conservative for positive restraint moment design, many designers have chosen early ages of continuity such as 28 days or even 17 or 14 days to ensure that the worst case positive moment is predicted.

**RESULTS AND DISCUSSION**

The final predicted positive restraint moments are plotted for each of the four different cases considered. Figure 1 shows the six plots for the 6 ft 0 in. slab widths and Figure 2 shows the six plots for the 10 ft 0 in. slab widths. Along the horizontal axis is the age at which the continuity connection was assumed to occur. Along the vertical axis is the expected positive restraint moment in kip-ft. The moment shown at each age of continuity does not necessarily occur at that time, but instead develops from that time to some “worst case positive” value at a later age. For many cases, the positive moment is shown as a negative moment. When this occurs, it means that the method predicts that a positive moment will not occur for that age of continuity. This moment shown, however, should not be considered the worst case negative moment. Also shown on each of the plots is the expected restraint moment due to the positive thermal gradient and the positive cracking moment capacity, both in units of kip-ft.

As can be seen in the plots, the predicted positive restraint moment due to the thermal gradient was similar in magnitude to the positive cracking moment capacity for all cases considered. The ratio of the positive cracking moment capacity to the thermal restraint moment ranges from 0.92 for a PCBT 45 with a 10 ft deck to 1.39 for a PCBT 93 with a 6 ft deck. In general, the girders with the smaller width decks showed larger ratios when compared to the same size girders with larger width decks. This makes sense since a larger deck width will allow for more forces to be created due to temperature gradients.

For all cases considered, it can also be seen that the predicted positive restraint moment due to the thermal gradient is greater in magnitude than most of the predicted positive restraint moments due to creep and shrinkage except at very early ages of continuity for a few situations. As the age of continuity increases, the predicted positive restraint moment due to creep and shrinkage eventually goes to zero and then becomes negative, indicating that a positive restraint moment due to creep and shrinkage will not occur. Therefore, for these ages of continuity, any positive restraint moment observed will more than likely be due to thermal effects instead of creep and shrinkage effects.

For all cases considered the PCA method predicted the highest, or most conservative, positive restraint moments when compared to the other three methods with ages of continuity of 28 days or greater. The RMCalc method and Method 1 predicted moments which were in fairly good agreement with each other for the later ages of continuity while Method 2
predicted moments that were generally in between the predicted moments due to the PCA method and the other two methods.

Figure 1 – Positive Restraint Moments for 6 ft 0 in. slabs

Comparing the two plots for the same girder size and slab widths, it can be observed that as the span length decreases (along with a decrease in the number of strands), that the predicted positive restraint moments due to creep and shrinkage generally decrease. This decrease is more evident for the girders with small slab widths, or closely spaced girders.
The ratio of the expected restraint moments to the expected positive cracking moment capacity were calculated for each of the cases considered and is shown in Figure 3. In order to present the data on one plot, the moment ratios were plotted against a ratio of the span length to the number of strands for each case. The first plot shows ratios determined for an age of continuity of 14 days and the second plot shows ratios determined for an age of continuity of 28 days. When the ratio became larger than 2.4, then a value of 2.4 was plotted. Also, when the predicted positive restraint moment became negative, indicating that
a positive moment will not occur, the ratio was plotted at a value of 2.4. The value of 1.2 times the cracking moment capacity is also plotted on each.

Figure 3 – Ratios of $M_{cr}$ to Predicted Moments for Cases Considered

The plots show that some values of the ratios of predicted positive cracking moment to the predicted positive thermal restraint moment fall below the 1.2 $M_{cr}$ line, indicating that there is a potential for the positive thermal restraint moments to be greater than 1.2 times the positive cracking moment capacity. Also for both ages of continuity, the PCA method predicts that positive restraint moments due to creep and shrinkage may exceed 1.2 times the positive cracking moment capacity for a number of cases considered. For the other three cases considered, there are a few ratios that fall below 1.2 $M_{cr}$ for an age of continuity of 14 days, but none that fall below 1.2 $M_{cr}$ for an age of continuity of 28 days.

CONCLUSIONS

1. For all cases considered, predicted positive thermal restraint moments are significant, ranging in magnitude from 0.72 to 1.02 (1/1.39 to 1/0.99) times the calculated positive cracking moment capacity.
2. For nearly all ages of continuity, especially for the later ages, the PCA method was the most conservative of the four methods, predicting the highest positive restraint moments due to creep and shrinkage.
3. For ages of continuity of approximately 60 days and longer, Method 1 compared well the RMCalc method in the prediction of positive restraint moments due to creep and shrinkage.
4. For span and strand arrangements typically used for design, as the span length and the number or strands decrease, the predicted positive restraint moments due to creep and shrinkage also decrease.
5. For a combination of early age of continuity and upper end range of span lengths and number of strands, three of the methods did predict ratios of $M_{cr}$ to predicted creep and shrinkage moments less than 1.2. This indicates that there are some design combinations that may require larger than 1.2 times the cracking moment.
REFERENCES


9. McDonagh, M., 2001, RMCalc, computer software, member Washington State Department of Transportation’s Alternate Route Project.


DESIGN EXAMPLE

GENERAL INFORMATION

For the adjacent typical section used in a two span continuous system with 100 ft spans, the following design example shows how restraint moments due to creep and shrinkage and restraint moment due to positive thermal gradient are calculated.

\[
k = 1000 \text{lb} \quad \text{kft} = k \cdot \text{ft} \\
\text{ksi} = \frac{k}{\text{in}^2} \quad \text{klf} = \frac{k}{\text{ft}}
\]

Girder: PCBT 45 \( f_c=8 \text{ ksi} \)  \( \text{EG} = 4578 \text{ ksi} \)

Noncomposite \( \text{IG} = 207300 \text{in}^4 \) \( y_{bot} = 22.23 \text{in} \)

Composite \( \text{IG}_{\text{comp}} = 419130 \text{in}^4 \) \( y_{bot_{\text{comp}}} = 32.26 \text{in} \)

Deck: 7 1/2 in. by 6 ft \( f_c=4 \text{ ksi} \)  \( \text{ED} = 3530 \text{ksi} \)

Strand: 7 wire, 1/2" diameter, Grade 270 \( \text{Aps} = 0.153 \text{in}^2 \) \( f_{\text{pu}} = 270 \text{ksi} \)

Release \( \text{fps}=0.75f_{\text{pu}} \) \( \text{Prel} = \text{Aps} \cdot f_{\text{pu}} \cdot 0.75 \) \( \text{Prel} = 31 \text{k} \)

Effective \( \text{fse}=0.60f_{\text{pu}} \) \( \text{Peff} = \text{Aps} \cdot f_{\text{pu}} \cdot 0.60 \) \( \text{Peff} = 24.8 \text{k} \)

Span Length: \( L = 100 \text{ft} \) 2 Span Continuous Unit

Given the following Strand Pattern for the 1/2 girder, with all dimensions in inches:
The End Rotation, $\theta$, due to the Effective Prestress Force can be calculated as:

$$\theta = \frac{16695453 \cdot k \cdot \text{in}^2}{EG \cdot IG}$$

Loading:
- Self Weight of Girder $\text{w}_{\text{self weight}} = 0.778 \text{klf}$
- Deck Weight $\text{w}_{\text{deck}} = 0.5625 \text{klf}$
- Noncomposite Dead Load $\text{w}_{\text{ncdl}} = 0.200 \text{klf}$
- Composite Dead Load $\text{w}_{\text{compl}} = 0.135 \text{klf}$
- Total Uniform Load $w = 1.6755 \text{klf}$

PCA METHOD

First, determine the moments unadjusted for creep effects. Moment distribution can be used to determine that the restraint moments for a two span unit are $3EI/L$ times the end rotation for the prestress force, $-1$ times the moment at center span for dead load, and $-1.5$ times the simple span moment due to differential shrinkage for the differential shrinkage moment. Figures in this method are from Reference No. 6.

Restraint Moment due to Prestress, $M_{ps}$:

$$M_{ps} = \frac{3EI \cdot G}{L} \cdot \theta$$

$$M_{ps} = 3478.2 \text{kft}$$

Restraint Moment due to Dead Load, $M_{dl}$:

$$M_{dl} = -1 \frac{w \cdot L^2}{8}$$

$$M_{dl} = -2094.4 \text{kft}$$

Restraint Moment due to Differential Shrinkage, $M_{s}$:

$$\varepsilon_s = .0006 \quad e = 12.74 \text{in} \quad t = 7.5 \text{in} \quad A_{\text{deck}} = 540 \text{in}^2 \quad E_{\text{deck}} = 3530 \text{ksi}$$

$$M_{s} = -1.5 \cdot \varepsilon_s \cdot E_{\text{deck}} \cdot A_{\text{deck}} \left( e + \frac{t}{2} \right)$$

$$M_{s} = -2357.5 \text{kft}$$

Adjust for 70% Humidity and Loading at 28 Days:

$$c_1 = 0.73 \quad \text{From Figure 10 for 70% Humidity}$$

$$\varepsilon_s \cdot c_1 = 4.38 \times 10^{-4}$$

$$c_2 = 0.41 \quad \text{From Figure 8 for Loading at 28 Days}$$

$$M_{s} = M_{s} \cdot c_1 \cdot c_2$$

$$M_{s} = -705.6 \text{kft}$$
The Adjustments for Creep are then determined:

The specific creep, from Figure 5, is determined as: 

\[ \varepsilon_c = 0.32 \times (10^{-6}) \]

\[ c_3 = 1.8 \quad \text{From Figure 6 for Release at 1 day} \]
\[ c_4 = 1.25 \quad \text{From Figure 7 for } v/s \text{ Ratio of } 3.4 \]

\[ \varepsilon_c = \varepsilon_c \cdot c_3 \cdot c_4 \cdot (1 - c_2) \quad \varepsilon_c = 4.248 \times 10^{-7} \]

Creep Coefficient, \( \phi \), equals: 

\[ \phi = \varepsilon_c \cdot E \cdot \frac{1000}{ksi} \]

The Final Restraint Moment is then determined as:

\[ M_{res} = (M_{ps} + M_{dl}) \cdot (1 - \exp(-\phi)) + M_s \cdot \frac{(1 - \exp(-\phi))}{\phi} \]

\[ M_{res} = 875.0 \text{kft} \]

COMPARISON METHOD # 1

Using ACI 209 to predict the Creep and Shrinkage of the Girder:

Creep: \( \nu_u = 1.50 \quad \text{At 28 days of continuity} \quad \nu_t = 0.63 \)

Therefore, the remaining creep is: 

\[ \nu_r = \nu_u - \nu_t \quad \nu_r = 0.9 \]

Shrinkage: \( \varepsilon_{shu} = 446 \times 10^{-6} \quad \text{At 28 days of Continuity} \quad \varepsilon_{sh} = 147 \times 10^{-6} \)

Therefore, the remaining shrinkage is: 

\[ \varepsilon_{shr} = \varepsilon_{shu} - \varepsilon_{sh} \quad \varepsilon_{shr} = 2.99 \times 10^{-4} \]

Ageing Coefficient: Table 5.1.1 of ACI 209, the Ageing Coefficient X is determined as:

For Prestress Force and Dead Loads \( X_1 = 0.72 \)

For Differential Shrinkage \( X_2 = 0.81 \)

Restraint Moment due to Prestress Force, Mps and Mpsloss 

Mps = 3478.2 kft

Using the provisions of AASHTO Standard Specifications, the moment due to the effective prestress force is adjusted for the percentage of losses which have occurred at the time of continuity. At 28 days continuity, 63% of the total losses have occurred.

Mps without losses: \( M_{ps1} = M_{ps} \cdot \frac{P_{eff}}{P_{prel}} \quad M_{ps1} = 4347.8 \text{kft} \)

Mps Losses: \( \Delta M_{ps} = M_{ps1} - M_{ps} \quad \Delta M_{ps} = 869.6 \text{kft} \)

For 63% losses at 28 day continuity, \( f_1 = 0.63 \)
Mps = Mps1 − f1·ΔMps  \[ Mps = 3800 \text{kft} \]

Mpsloss = (1 − f1)−ΔMps  \[ Mpsloss = −321.7 \text{kft} \]

Restraint Moment due to Dead Load, Mdl:
\[ \text{Mdl} = −2094.4 \text{kft} \]

Restraint Moment due to Differential Shrinkage, Ms:
\[ \varepsilon_{shudeck} = 520 \cdot 10^{-6} \quad \varepsilon_{sh} = \varepsilon_{shudeck} − \varepsilon_{shr} \quad \varepsilon_{sh} = 2.21 \times 10^{-4} \]
\[ Ms = −1.5 \cdot \varepsilon_{sh} \cdot E_{deck} \cdot A_{deck} \cdot e^t \left( \frac{1}{1 + X1 \cdot \varepsilon_{r}} \right) \]
\[ Ms = −868.3 \text{kft} \]

The Final Restraint Moment is determined as:
\[ M_{res} = \left( \frac{\varepsilon_{r}}{1 + X1 \cdot \varepsilon_{r}} \right) \cdot (M_{ps} + M_{mdl}) + \left( \frac{1}{1 + X2 \cdot \varepsilon_{r}} \right) \cdot M_{psloss} + \left( \frac{1}{1 + X1 \cdot \varepsilon_{r}} \right) \cdot (M_{s}) \]
\[ M_{res} = 205.2 \text{kft} \]

COMPARISON METHOD #2

Using the MC-90 to predict the Creep and Shrinkage of the Girder and the Deck

Creep: Girder: For 28 day Continuity  Occurred:  \( \phi_{go} = 0.692 \)
Remains:  \( \phi_{gr} = 1.232 \)

Deck: Ultimate Creep  \( \phi_{d} = 2.605 \)

Shrinkage: Girder:  Occurred:  \( \varepsilon_{cso} = 60.5 \cdot 10^{-6} \)
Remains:  \( \varepsilon_{csr} = 321 \cdot 10^{-6} \)

Using the provisions of AASHTO Standard Specifications, and subtracting the contribution of elastic shortening losses to the total prestress losses, an initial prestressing force after elastic shortening losses can be determined as a percentage of the effective prestress force. The initial PS force is 1.103 times the effective PS force.
\[ c = 1.103 \]

Restraint Moment due to Prestress force and Prestress losses, Mps:

The total end rotation at an interior joint, \( \Delta D \) due to PS force is:
At an effective age of release of strands equal to 10.21 days
\[ \text{Ageing Coefficient } X1 \text{ is: } X1 = \frac{\sqrt{10.21}}{1 + \sqrt{10.21}} \]
\[ X1 = 0.762 \]
Effective Modulus of Elasticity for the Girder is:

\[
E_{\text{Geff}} = \frac{EG}{1 + X_1 \phi_{gr}}
\]

\[\Delta D_{ps} = c \cdot \theta \cdot (-2) \cdot \phi_{gr}\]

\[\Delta D_{psloss} = (c - 1) \cdot \theta \cdot 2 \cdot (1 + X_1 \phi_{gr})\]

The age adjusted Flexibility Coefficient, \(f\), for the Two Span Structure is:

\[
f = \frac{2 \cdot L}{3 \cdot E_{\text{Geff}} I_G}
\]

The restraint moment, \(F_{ps}\), due to the initial prestress force and prestress loss is:

\[
F_{ps} = \left(\frac{\Delta D_{ps} + \Delta D_{psloss}}{f}\right) \cdot (-1)
\]

\(F_{ps} = 2079.9\ \text{kft}\)

Restraint Moment due to Dead Load, \(M_{dl}\)

\(M_{dl} = -2094.4\ \text{kft}\)

Total Rotation, \(\Delta D_{dl}\), due to the Dead Load Moment at an interior joint:

\[
\Delta D_{dl} = \frac{L \cdot 2 \cdot M_{dl}}{3 \cdot E G \cdot I G} \phi_{gr}
\]

Restraint Moment, \(F_{dl}\), due to this rotation is:

\[
F_{dl} = \frac{\Delta D_{dl}}{f}
\]

\(F_{dl} = -1331.2\ \text{kft}\)

Restraint Moment Due to the Differential Shrinkage

For a given deck shrinkage of: \(\varepsilon_{sh\text{deck}} = 440 \cdot 10^{-6}\)

Remaining Shrinkage is: \(\varepsilon_{sh} = \varepsilon_{sh\text{deck}} - \varepsilon_{cst}\)

\(\varepsilon_{sh} = 1.19 \times 10^{-4}\)

\(M_s = -1 \cdot \varepsilon_{sh} \cdot E_{\text{deck}} \cdot A_{\text{deck}} \left( e + \frac{t}{2} \right)\)

\(M_s = -311.7\ \text{kft}\)

The End rotation, \(\Delta D_s\), due to this moment is:

\[
\text{For an effective age of 28 days } \quad X_2 = \frac{\sqrt{28}}{1 + \sqrt{28}} \quad X_2 = 0.841
\]

\[
\Delta D_s = \frac{M_s \cdot L \cdot \left(1 + X_2 \cdot \phi_{gr}\right)}{E G \cdot I G_{\text{comp}}}
\]

The age adjusted Flexibility Coefficient, \(f\), for the Two Span Structure is:

\[
f = \frac{2 \cdot L}{3 \cdot E_{\text{Geff}} I G_{\text{comp}}}
\]
The Restraint Moment, $F_s$, due to the differential Shrinkage is:

$$F_s = \frac{\Delta D_s}{f} \quad F_s = -491.2 \text{kft}$$

Therefore, the total Restraint Moment is:

$$M_{res} = F_{ps} + F_{dl} + F_s \quad M_{res} = 257.6 \text{kft}$$

RMCALC METHOD

Average values from the above three methods were used for input for the RMCalc Method.

For the ultimate creep coefficient of the girder:

$$\frac{1.94 + 1.50 + 1.92}{3} = 1.79$$

For the ultimate shrinkage strain in the girder:

$$\frac{438 + 446 + 382}{3} = 422 \text{ microstrain/in.}$$

For the ultimate shrinkage strain in the deck:

$$\frac{520 + 440}{2} = 480 \text{ microstrain/in.}$$

$$M_{res} = 411.2 \text{kft}$$

THERMAL GRADIENTS

To simplify calculations, champers were ignored, and the bolster was modified so that the following equivalent section could be used for the determination of the restraint moment.
The moduli of elasticity for the deck and the girder are:

\[
ED = 3530 \text{ ksi} \quad EG = 4578 \text{ ksi}
\]

Using an \( \alpha \) for the concrete of 0.000006/Degree F, the stress at points 1 and 2 are:

\[
\text{ORIGIN} = 1 \\
\alpha = 0.000006 \quad T_1 = 41 \quad T_2 = 11 \\
\sigma = ED \cdot \alpha \cdot T \quad \sigma_2 = 0.2 \text{ ksi}
\]

The force in Section Number 1 can be determined as:

\[
b = 72\text{ir} \quad h = 4\text{ir} \\
F = b \cdot h \cdot \sigma_2 \quad F = 67.1 \text{ k}
\]

This force acts at a distance from the centroid of 18.24 in. The Moment due to this force is:

\[
d = 18.24\text{in} \\
M = F \cdot d \quad M = 1223.9 \text{ kin} \quad \text{or} \quad M = 102 \text{ kft}
\]

The remaining sections are:

<table>
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<tr>
<th>Point Number</th>
<th>Dist. From CG, in.</th>
<th>Temp., °F</th>
<th>Stress, σ ksi</th>
<th>Section Number</th>
<th>Force, kips</th>
<th>Moment, kip-in.</th>
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<td>P1</td>
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<td>0.868</td>
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<td>67.1</td>
<td>1223.9</td>
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<td>P2</td>
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<td>2</td>
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<td>1728.4</td>
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<td>0.168</td>
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Summation: 3534.2

Therefore, the total restraint moment is: \( M_{\text{thermal}} = 3534.2 \text{ kin} \) \quad \text{or} \quad M_{\text{thermal}} = 294.5 \text{ kft}

Due to continuity, the final restraint moments is: \( M_{\text{thermal}} = 1.5 \cdot M_{\text{thermal}} \)

\( M_{\text{thermal}} = 441.8 \text{ kft} \)
Appendix D: Shop Drawing from Precaster
Appendix E: Concrete Test Results

Summary of Girder Concrete

<table>
<thead>
<tr>
<th>Age of Concrete Days</th>
<th>Compressive Strength f’c, ksi</th>
<th>Modulus of Elasticity/1000 ksi</th>
<th>Split Tensile Strength ft, ksi</th>
<th>Beam Rupture Strength fr, ksi</th>
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<td>0.5</td>
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Summary of Deck Concrete

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<th>Split Tensile Strength ft, ksi</th>
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Appendix F: Miscellaneous Calculations

Determine the Capacity for Positive Moment for Extended Strands

\[
\begin{align*}
\text{Le} & := 24\text{in} & \text{Embedment Length} \\
n & := 12 & \text{Number of Prestress Strands} \\
\text{Astrand} & := 0.153\text{in}^2 & \text{Area of Prestressed Strand} \\
\text{Aps} & := n \cdot \text{Astrand} & \text{Total Area of Prestressed Strands} \\
\Phi & := 0.9 & \text{Understrength Factor} \\
f_c & := 4000\text{psi} & \text{Concrete Strength} \\
b & := 72\text{in} & \text{Width of Compression Face} \\
k & := 1000\text{lb} & \text{Unit Conversion} \\
\text{ksi} & := 1000\text{psi} & \text{Unit Conversion} \\
\text{kin} & := \text{ksi-in}^3 & \text{Unit Conversion} \\
kft & := \text{kin-in} & \text{Unit Conversion} \\
\text{td} & := 7.5\text{in} & \text{Deck Thickness} \\
\text{tb} & := 2.5\text{in} & \text{Bolster Thickness}
\end{align*}
\]

For PCBT-45 with 12 Strands Extended at 9" out plus 15" up equals 24" embedment:

\[
\begin{align*}
\text{fpu} & := \frac{(\text{Le} - 8\text{in})}{0.163} \text{in} & \text{fpu} = 98.16\text{ksi} \\
\text{a} & := \frac{(\text{Aps} \cdot \text{fpu})}{0.85f_c - b} & a = 0.736\text{in}
\end{align*}
\]

\[
\begin{align*}
h & := 45\text{in} & \text{dps} := h - 2\text{in} + \text{td} + \text{tb} &\text{dps} = 53\text{in} \\
\text{Mu} & := \Phi \left[ \text{Aps} \cdot \text{fpu} \left( \text{dps} - \frac{a}{2} \right) \right] & \text{Mu} = 8.537 \times 10^3 \text{kin} \\
\text{Mpos} & := \text{Mu} & \text{Mpos} = 711.402\text{kft}
\end{align*}
\]

Capacity of Section with U-Shaped Mild Steel Reinforcing Bars

| 4-6/6U |  
|---|---|---|---|---|---|---|---|---|---|---|
| Mark | Height, in | Bolster, in | Thick, in | Width, in | Htotal, in | Fc, ksi | Beta | Phi*Mn |  
| PCBT-45 | 45 | 1.5 | 7.5 | 72 | 54 | 4 | 0.85 |  
| Level | Abar | Number | Ybot, in | d, in | d-a/2, in | Strain | Check | Atotal | T.C, kips | a, in | c, in | M, kin |  
| 1 | 0.44 | 4 | 2.125 | 51.875 | 51.44385 | 0.150326 | OK | 1.76 | 452.7036 |  
| 2 | 0.44 | 4 | 7.375 | 46.625 | 46.19363 | 0.134809 | OK | 1.76 | 406.5039 |  
| Phi*Min | 773.3 |  

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Appendix G: Additional Testing Results

Thermocouple Readings - VT1

Thermocouples Readings - VT3
Cumulative Strains - VT1

Cumulative Strains - VT2

Hours After Casting

Microstrain

VWG 1-1
VWG 1-2
VWG 1-3
VWG 1-4
VWG 1-5

VWG 2-1
VWG 2-2
VWG 2-3
VWG 2-4
VWG 2-5
Cumulative Strains - VT5

Cumulative Strains - VT6

Hours After Casting

Microstrain

VWG 5-1
VWG 5-2
VWG 5-3
VWG 5-4
VWG 5-5

VWG 6-1
VWG 6-2
VWG 6-3
VWG 6-4
VWG 6-5
Appendix H: Deck Damage Plans
Test 1 Deck Cracking

Active End

10'-0"

10'-0"

Max Width
0.010 in.
Linear Ft.
73.7

Passive End

6'-0"

Test 1 Deck Cracking
Scale: 3/8" = 1'-0"
Test 2 Deck Cracking

Active End

Max Width
0.013 in.
Linear Ft.
45.3

Test 2 Deck Cracking
Scale: 3/8" = 1'-0"
Test 3 Deck Cracking

Active End

Passive End

Max Width
0.040 in.
Linear Ft.
25.6

Scale: 3/8" = 1'-0"
Appendix I: Recommended Continuity Detail
Typical Test Section
Scale: 1/2"=1'-0"

Prestressing Strand Pattern
Scale: 1/2"=1'-0"

"As Built"
4-#6 (180 U-Bars)
Bend at 4 1/2" Diam.
Place adjacent to strands as shown.

End View
Scale: 1/2"=1'-0"

Test Option 2 (TO2)

"As Designed"

"As Tested"

Side Elevation
Scale: 1/2"=1'-0"
Typical Section

Test #2

"As Built"
Vita

Charles David Newhouse was born in Salem, Missouri on December 29, 1969. He moved to Gloucester, Virginia when he was under 3 years old and lived there until graduating from high school in 1988. He attended Virginia Polytechnic Institute and State University from 1988 until 1993 and received B.S. and M.S. degrees in Civil Engineering. For his M.S. degree, he worked for Dr. Weyers on a Thesis entitled *Corrosion Rates and the Time to Cracking of Chloride Contaminated Bridge Components*. Following graduation in 1993, he began work as a consulting structural engineer in Norfolk, Virginia for MMM Design Group. After over 8 years working as a consultant, he returned to Virginia Polytechnic Institute and State University in the summer of 2002 and has worked for Dr. Roberts-Wollmann on this project since that time.