INTERIM REPORT

PROJECT 2000-23
IMPROVEMENT OF CONTINUITY CONNECTIONS OVER FIXED PIERS

SUMMARY
This interim report consists of two stand-alone parts: 1) Recommendations for changes to existing NJDOT design specifications for continuity connections, and 2) A research progress report including discussion of the NJDOT design approach, a summary of the questionnaire distributed to all state DOTs, field measurements conducted under this study, and finite element modeling/preliminary results.

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PART I

RECOMMENDATIONS FOR CHANGES TO EXISTING NJDOT DESIGN SPECIFICATIONS FOR CONTINUITY CONNECTIONS

The following recommendations are provided in order to improve the standard continuity design for live load as illustrated on Guide Sheet Plates 3.10-15 to 3.10-18 (NJDOT Bridge and Structures Design Manual, 3rd Edition, 1998). These recommendations are based on a literature search, evaluation of the responses to a questionnaire (conducted under this study) by 27 departments of transportation, and structural analysis and evaluation of existing continuity design vis-à-vis expected performance. Note that the 1998 NJDOT Design Manual uses metric units. However, to simplify adoption of these recommendations into the next Design Manual, which will use English system, this is the primary system in the following discussions.

The recommendations are listed below and each is discussed in more details in the following pages.

1. Eliminate anchor bolts (dowels) or sheath them to prevent bonding with diaphragm concrete.
2. Cap top of the anchor bolts (dowels) with Styrofoam.
3. Debond girders from the diaphragm or do not embed them at all.
4. Employ a balanced design concept between dead load and prestressing forces/deflections.
5. Provide two rebars along the pier cap within the positive reinforcements.
6. Consider a wider connection to ease construction and to avoid embedment of the girders in the diaphragm.
1. Eliminate anchor bolts (dowels) or sheath them to prevent bonding with diaphragm concrete.

Figure I.1 is taken from the NJDOT Bridge and Structures Design Manual. This design includes one 1.25"-diameter 2-ft long anchor bolt per line of girder (beam). Apparently this is to provide a nominal resistance in the horizontal plane although it is not designed for any specific force. It should be also noted that a keeper block is located at the center to provide seismic (lateral) resistance in the transverse direction.

If the anchor bolt is bonded to the diaphragm concrete it will provide rotational resistance at the continuity joint. Tensile force in the anchor bolt and bearing (compressive) force form a couple with very small lever arm. Thus, for a small load within the span a very large tensile force will develop in the anchor bolt. This force easily exceeds the pull out capacity of the anchor bolts causing cracking and damage to diaphragm concrete. This is further discussed in Part II. Therefore, it is recommended that the anchor bolt be eliminated all together or it must be sheathed. It can be eliminated because in the transverse direction the keeper block will provide resistance to lateral forces. In the longitudinal direction due to continuity of the bridge, span fall off is not an issue. If the design intension is to engage the pier as a seismic resistance element in the longitudinal direction then other options can be pursued (such as the use of keeper blocks). If the decision is to still use the anchor bolt then it should be designed accordingly and probably more than one anchor bolt per line of girder will be needed. In such case they must be sheathed. It is important for the design not to restrain rotation of the joint.

The anchor bolt can be sheathed with galvanized steel pipe (Figure I.2) or PVC pipe (as used by Tennessee Department of Transportation and discussed in Part II) or similar approach. Some states, such as New York, place the dowel in the girder using prefabricated anchor dowel hole and fill material.

Figure I.1 Continuity joint details (New Jersey)  
Figure I.2 Bar sheath and Styrofoam cap (Michigan)
2. Cap top of the anchor bolts (dowels) with Styrofoam.

Gravity forces should be transferred to the pier through the bearing pads and not the anchor bolt. If the superstructure bears on the anchor bolts it can further raise the possibility of damage to the diaphragm concrete. Therefore, the top of the anchor bolts (dowels) should be covered with flexible material, such as Styrofoam. This will allow downward deflection and bearing against the pads. Figure I.2 shows the 2” (50 mm) Styrofoam cap used by Michigan DOT. Louisiana uses a 1/2” (12 mm) cap while the State of Tennessee uses a 3”x3”x3” Styrofoam block.

3. Debond girders from the diaphragm or do not embed at all.

As it was mentioned before, it is important that the continuity connection allows for rotation of the joint. The existing design approach in New Jersey is to embed the girders in the diaphragm (see Figure I.1). This will restraint the rotational capacity of the joint, which can exert unanticipated forces on the connection. Therefore, it is recommended to apply bond breaker around the girder surface in contact with the diaphragm. Michigan observed problems using paraffin wax for this purpose and switched to roofing felt as a bond breaker. Illinois also uses roofing felt for this purpose. Another option is to not embed the girders in the diaphragm. This will require a cap pier of a larger width. The State of Tennessee uses this approach (see recommendation number 6).

4. Employ a balanced design concept between dead load and prestressing stresses/deflections.

A simply supported beam under dead load will have a moment diagram as shown in Figure I.3a, which will cause downward deflection. The qualitative moment diagram due to prestressing is shown in Figure I.3b. This moment causes the beam to camber upward, and creep will result in additional upward deflection. Once continuity is established and rotation is restrained, a positive moment will develop at the joint that may be large enough to cause cracking. The State of Tennessee’s practice is to not allow downward deflection under total dead load. However, perceived conservatism in design can easily translate this into designs with significant upward camber. For example, the State of Alabama has observed “upward deflection at mid-span (greater than predicted downward live load deflection)” that results in development of a significant level of stresses at the joint. Therefore, it is recommended to set a limit on the upward deflection too. This can be achieved by either specifying a limit on the net camber or more rationally by minimizing the difference between moments from dead load and prestressing force (shaded area in Figure I.3c). A more exact analysis will require consideration of shrinkage stresses too. This will minimize/eliminate bending creep (only axial creep), consequently positive restraint moment will be minimized or eliminated. It is not expected that this recommendation will affect live load deflection requirements. The intension is to reduce conservatism. It should also be noted that AASHTO LRFD gives more flexibility to designers on deflection (i.e., deflection and depth limitations are optional).
5. Provide two rebars along the pier cap within the positive moment reinforcements.

Detail “Y” for positive restraint moment connection (per NJDOT Design Manual) is shown in Figure I.4a. As it can be seen, there is only one rebar within the positive moment reinforcement. It is recommended to use two rebars, as shown in Figure I.4b, in order to provide better resistance to crack growth. The same is the case with detail “X” (not shown).

Figure I.3 Moment diagrams due to dead load and prestressing forces, and net result.

Figure I.4 Positive Restraint Moment Connection (New Jersey)
6. Consider a wider connection to ease construction and to avoid embedment of the girders in the diaphragm.

A continuity connection as shown in Figure I.5 will have several benefits by calling for a wider diaphragm and cap beam. This is the practice of the State of Tennessee. It will eliminate the need for embedment of girders. It will also allow for larger longitudinal spacing between the girders that will result in ease of construction and less reinforcement congestion. Of course, a drawback is larger pier cap beam. Positive moment continuity also does not seem to be as rigid as current approach. Adding one rebar at each bend along the diaphragm, similar to item 5, can enhance rigidity.

Figure I.5 Wider continuity connection similar to that used by the TNDOT
Introduction

Since early 1950’s, application of precast, prestressed I-girders to bridges has become widespread. In the United States, the National Bridge Inventory indicates that about 50 percent of bridges constructed over the past decade use prestressed concrete girders. To improve structural efficiency of multi-span simply supported bridges, partial and/or full continuity is provided through cast-in-place concrete diaphragm and decks. These bridges are known as Simple-Span Precast Prestressed Bridge Girders Made Continuous. Typical construction of this class of bridges includes the following steps (Figure II.1):

1. Erecting and aligning precast prestressed girders.
2. Connecting positive reinforcement (for some states). For others, such as NJ, positive reinforcements do not require attachment (they simply overlap).
3. Installing diaphragm and deck reinforcement.

Figure II.1 Construction sequence, time-dependent moment, and positive reinforcement details (a and b from reference [1]).
In addition to structural advantage of this kind of connections (i.e., continuity under additional dead load and live load), they are also beneficial from maintenance point of view by eliminating expansion joints. Maintenance and repair costs of open joints are major problems associated with maintaining highway bridges. However, as discussed in the original proposal, continuity connections also have their own structural, construction, and maintenance shortcomings that are the subject of this research project.

A major structural issue with continuity connections is uncertainty about the degree of continuity. These bridges are designed to have a net upward camber as shown in Figure II.1a. Time dependent effects (mainly creep of concrete), demand additional upward deformation and rotation at the ends. However, continuity connection restrains rotation. This causes development of positive restraint moment in the girders over the piers as shown in Figure II.1b. This moment can initiate cracks, which not only impair the aesthetics of the bridge but also cause corrosion of the reinforcement in the diaphragms. To provide negative moment continuity the cracks must close first, which needs relatively large live loads. As a result, the continuity of such bridges ranges from 0% to 100%, depending on loading condition, construction sequence, material properties of the concrete and reinforcement, and structural parameters such as span length, girder geometry, etc. These problems (uncertainty in degree of continuity, difficulty in construction, continuity for only a portion of total load, etc.) are common to design details used by many states and will be addressed through development of a “new connection” under this study. However, improvement to existing design practice in the State of New Jersey is another important objective of this study, which was the subject of the first phase of the work. Specific recommendations are provided as a companion to this progress report (see Part I). The following sections include evaluation of current details and expected versus actual performance, summary presentation and discussion of the results of a questionnaire distributed to other DOTs, in-field measurements, finite element modeling/results, and brief discussion of research plan for the next phase. Within the presentation of results, the reasons for recommendations made under Part I of this report are also highlighted.

**New Jersey Details and Possible Performance**

Although there are significant differences in details of continuity connections among DOTs in the US, the general design approach for positive reinforcement is similar among many states. It consists of extending prestressing tendons (or embedded reinforcement bars) into the diaphragm and bending them upward to form a U-shape for anchorage purposes. The positive reinforcement is not explicitly designed for any load and they are more for “integrity” purposes. Many engineers actually view the entire continuity connection itself as a means to eliminate an open joint rather than an effort to improve structural efficiency.

Figure II.2 shows a top view along with a typical cross section at a girder line per NJDOT’s design manual. Details of the connection include a 1.25” (32 mm) diameter anchor bolt per beam line that is 2’-0” (600 mm) long. At the continuous end of a multi-span beam rotation is allowed and there is no transfer of moment to the support (i.e., moment in the beam is constant at a section on the support). Figure II.3 shows the moment diagram for a 2-span
continuous beam loaded with a concentrated load at the center of one of the spans. However, if the connection provides resistance to rotation then part of the moment is transferred to the support and the moment diagram will approach that of a fixed end beam. Figure II.4 shows the reactions and moment diagram for a continuous 2-span beam with an internal support idealizing the details of Figure II.2. That is, there are double pins at the internal support where one support represents the neoprene pad adjacent to the loaded span, which will be in compression. The other support models the anchor bolt, which can provide tensile resistance when bonded to the concrete. This couple will provide resistance to rotation; however, since the lever arm is very small large forces will be required in order to balance the moment due to the concentrated load applied at the center of the beam. For the example shown, a 20 kip concentrated load on a 120-ft span will generate over 172 kips of force in the anchor bolt. This will easily exceed the pull out capacity of the bolt, thus, damaging the diaphragm. Therefore, it is recommended that the anchor bolt(s) be eliminated or sheathed if they are needed. A review of the design approach used by other states revealed that several states do indeed sheath the anchor bolts. Based upon discussion with designers, it appears that it is done for a different reason. The reason being the ability to easily jack the girders should there be such a need.

Apparently resistance to lateral load (such as seismic or possibly wind) is a primary function of the anchor bolt. However, there are other elements (such as keeper blocks) or options to employ in resisting lateral forces. It should also be noted that one anchor bolt alone is not adequate for this purpose and depending on the level of lateral forces additional anchor bolts should be placed in the diaphragm. But as it was mentioned they should be sheathed. Various materials have been used by other states to cover the anchor bolt. For example, the State of Michigan uses galvanized steel pipe (Figure II.5) while the State of Tennessee uses a PVC pipe.

Figure II.2 Sections from continuity joint per NJDOT Design Manual for Bridges and Structures (1998).
Figure II.3 Reactions and moment diagram for a two span continuous beam loaded with a concentrated load at the center of left span: **single support at the center.**

Figure II.4 Reactions and moment diagram for a two span continuous beam loaded with a concentrated load at the center of left span: **double support at the center.**
Another possible source of damage to the diaphragm could result from bearing of the superstructure weight on the anchor bolts. The design must ensure gravity loads are transferred to the columns through the bearing pads and not the anchor bolt. To achieve this it is recommended to encase the top of the anchor bolt with Styrofoam, as shown in Figure II.5.

The State of Tennessee uses a 3”x3”x3” block of Styrofoam to cap the anchor bolts as explained in the following section, while the State of Louisiana uses a 1” (12 mm) thick compressible cap.

As it was mentioned before, for the superstructure to behave as continuous beam it is important that the continuity connection does not restrain rotation. Embedding girders in the diaphragm (see Figure II.2) will restrain the ability of the beam to rotate. Similar to restraining effect of the anchor bolt this can also damage the diaphragm and possibly the girder itself. Therefore, it is recommended to debond girders from the diaphragm or do not embed them at all. Several states, such as Illinois and Michigan, use roofing felt for debonding, while the State of Tennessee does not embed the girders at all. A disadvantage of the latter option is that it will result in a wider pier cap beam.

**Figure II.5** Bar sheath and cap
Styrofoam (Michigan).
Literature Review

A complete review of available literature on the subject will be provided in the final report. The literature available is quite limited and can be simply categorized into three groups/periods. The first part of the published work deals with the original work initiated by the Portland Cement Association (PCA) in the 60s, which resulted in the design guidelines that are still used by AASHTO. During the initial period of application of precast, prestressed girders (i.e., until early 1960’s) these systems were designed as simply supported spans. To better understand this and to develop design procedures, in early 1960’s PCA conducted an extensive experimental and analytical investigation of this class of bridges [2-5]. As a result of this research PCA developed a design procedure [6] which provided the means for recognizing the structural continuity obtained in bridges built with precast, prestressed girders and cast-in-place concrete deck starting in the 1971 Interim AASHTO Specifications. An important development under the PCA studies was the recommendation for the design of positive moment reinforcement.

To address uncertainties associated with behavior and design of the continuity joints, in the late 80s the Transportation Research Board initiated NCHRP Project 12-29, “Design of Simple-Span Precast Prestressed Bridge Girders Made Continuous.” The findings of this project, which constitute the second group of work on the subject, are reported in the National Cooperative Highway Research Program Report 322 – NCHRP Report 322 [1]. A major recommendation of this report is elimination of the positive moment connection, thus, eliminating one of the most difficult and expensive problems associated with simple-span precast prestressed bridge girders made continuous. It is reported that “The effect of the lack of apparent full continuity, caused by not providing positive reinforcement in the diaphragm and allowing the bottom diaphragm crack to open, is virtually balanced by the increased positive restraint moment that develops when positive reinforcement is provided. When negative restraint moments develop, positive moment reinforcement is in the compression zone and offers no structural advantage. Therefore, the resultant mid-span moments, which include moments due to dead load, restraint moments due to creep and shrinkage, and live load plus impact moments, are virtually independent of the area of positive reinforcement provided in the diaphragm connections at the supports.” The findings and design guidelines developed under this study have been put in the form of specifications for possible incorporation into the AASHTO Specs. However, they were not adopted.

In 1999 NCHRP sponsored another project (Project 12-53), entitled “Connection Between Simple-Span Precast Concrete Girders Made Continuous.” This work is being conducted at University of Cincinnati with Professor Richard Miller as the principal investigator. It is essentially the start of the third period of the continuing work on the subject over the past three decades. It is a three-year project and still active. However, some preliminary results have been published [7]. With regard to positive moment reinforcement although it agrees with NCHRP 322 that total mid-span moments are virtually independent of the amount of reinforcement, positive moment reinforcement is recommended for durability and structural integrity. The work plan for the remainder of the project includes full-scale tests considering
parameters such as bent strands or bent bars as positive moment reinforcement, with or without girder embedment into the diaphragm, etc.

In summary, it appears that the three main research works on the subject, performed at different periods over the last four decades, resulted in recommending, revoking, and again recommending the use of positive moment reinforcements. This reflects the need for additional research work on the subject and further highlights why most states, if not all, use this type of connection more for maintenance purposes (to eliminate an open joint) than structural efficiency. Furthermore, as it is discussed in the next section, most states use the positive moment reinforcement for additional durability and integrity than any specific structural objective. But it appears that positive moment reinforcement may be causing additional maintenance problems while its contribution to structural integrity is marginal or only “perceived”.

It should be noted that the brief literature review presented deals with the type of connection where continuity is provided through cast-in-place concrete diaphragm and decks. Connections that utilize post-tensioning or mechanical systems to achieve continuity (often full continuity) have also been studied over the past several decades (e.g., [8]). However, due to cost and/or the construction skills required they are not as popular and not within the scope of this study.

**Questionnaire Results**

To gage the experience of other highway agencies with design of continuity connections and to determine their practice vis-à-vis the NCHRP Report 322 recommendations, a questionnaire was developed and distributed to all highway agencies. We have received responses from 26 states (AK, AL, CA, CO, CT, GA, HI, ID, IA, IL, KS, LA, ME, MI, MT, NE, NH, NV, NY, PA, SC, TN, TX, UT, WA, WI). Individual responses along with a copy of the questionnaire will be attached to the final report as an appendix. Here an overview of the responses that will be presented:

?? Maintenance. It is common in the replies to our query to have people respond that they use the continuity connection because it allows them to do away with expansion joints.

?? Constructability. Many of the responses cite difficulty in making the continuity connection when they have to deal with projecting reinforcing bars and prestressing strands.

?? Satisfaction. In general the response has been that the DOTs are happy with this type of construction. On the other hand, Alabama, with some 200 bridges experience, says that they no longer use this type of construction. They cite serious thermal stress problems that led to cracking of the continuous joints. It has been suggested that Alabama’s problems are the result of poor detailing.

?? Cracks. Several states cited problems with cracks forming between the girder and the support diaphragm.

?? Analysis. It is common to assume simply supported for both dead load and live load, essentially not taking advantage of the structural benefit of continuity. This reinforces the fact that the connection is used more to eliminate a joint than anything else.
Degree of continuity. None of the states have conducted any experimental work to determine the actual degree of continuity.

Positive reinforcement. Many of those states that employ the continuity connection use a minimum amount of positive reinforcements ($1.2M_{cr}$). However, this reinforcement is not designed for any specific load. It is believed that it will enhance “structural integrity” and provide “redundancy.”

Seismic issues. Several states cite the continuous connection as helping them with seismic problems.

The details of positive reinforcement, if used, is similar to that used by NJDOT. However, the state of New York uses welded bar connection for I-Beam details. NYDOT also places the anchor bolts in the girders using prefabricated holes. A couple of states (Connecticut and Texas) do not use this type of bridge at all, while several other states (GA, HI, ME, NH, NV) have only one or two bridges with continuity connections. It should be noted that there are states such as Michigan and Utah that do not use positive reinforcement in their continuity joints at all and are satisfied with their performance. However, both states design their bridges as simply supported for both dead load and live load. The States of Alabama, Colorado, Illinois, Michigan, Pennsylvania, South Carolina, Tennessee, and Wisconsin have the most experience with this type of bridge. Of these states, Alabama reports dissatisfaction with the connection due to cracking. ALDOT practice now is to not provide any positive reinforcement. To eliminate open joints, some spans are designed with “only the slab poured continuous with no connection of the girder ends.” The state of Georgia limited the use of continuity connections because of difficulty in construction and cracking of “the end of the heavily reinforced areas of beams (about ten feet from the beam ends on either side of the joints).” It should be noted that during field surveys conducted under this study, in a few cases “cracks” were observed at the ends of some girders. That is, vertical orange/red lines spaced uniformly were observed at the end of some girders that started from the bottom and propagated upward into web. Furthermore, as it will be discussed in the following section damage to girders at the top of bearing pads was observed during a recent site visit. The state of Iowa extends and bends the top reinforcement too in order to increase the integrity of the structure. But they too still design the beams as simply supported for all loads because of “some problem with cracking” and “because of concerns about how much continuity” is achieved. The State of Tennessee is among the most experienced state with very satisfactory experience with this type of bridges. TNDOT uses a wider diaphragm in order to prevent overlap of positive reinforcement and to not embed the girders. They explicitly design the anchor bolts for seismic loads, however, they do sheath the bolts to prevent bonding with diaphragm concrete and to allow rotation. TNDOT does not agree with NCHRP Report 322 recommendations. On the other hand, the Michigan DOT, which also has extensive experience with the use of these joints, does agree with those recommendations and does not provide positive reinforcements. But as it was mentioned, they design the beams as simple span for both dead and live loads. TNDOT also specifies minimum age of 90-days for the girders prior to establishment of continuity.

In summary, despite general similarity, the design approach and details of the continuity connection varies significantly among various states. A copy of the replies to the questionnaire for all states will be provided as an appendix to the final report.
Field Measurements

As it was mentioned before, the level of continuity can vary from 0 to 100 percent depending on many factors such as girder age at the time of erection, amount of positive reinforcement in the diaphragm at piers, creep and shrinkage properties of girder and deck concrete, construction sequence of deck and diaphragm, level of live load, etc. There has not been any attempt during previous research work to determine the degree of continuity under service load. Therefore, one of the tasks under this study is to instrument bridges to determine the actual degree of continuity.

The research team visited many bridges on a list provided by NJDOT to identify the most suitable ones for instrumentations. Among factors important to the selection process were ease of access to the girders and minimum or no closure of traffic. In general it was very time consuming and difficult to identify a suitable bridge. During the initial phase of this study two bridges were selected for instrumentation. Subsequently, based on presentation of progress reports and discussions with NJDOT staff, it was determined that it is valuable to the research if additional bridges are instrumented. Therefore, another bridge was identified for instrumentation to assess its response in order to determine its degree of continuity. Thus, a total of three bridges have so far been instrumented and responses have been measured. These are briefly discussed in the following paragraphs.

I-287 Over Darlington Avenue: This is a multi-span precast prestressed bridge with seat type abutments and cast in place deck. Southbound bridge has four spans while the northbound bridge has three spans. In the northbound bridge, as shown in Figure II.6, continuity connection is provided only at one of the piers (identified as a fixed joint). The northbound bridge was instrumented. Instrumentation included placement of strain gages at three locations: end of B10 girder at the continuity joint, end of B16 girder at the continuity joint, and center of B16 girder. At each end location there are three strain gages: one at the bottom of the bottom flange and two at the opposite sides of the top girder flange.

![Figure II.6 I-287 Over Darlington Avenue](image-url)
At the center location there is only one strain gage at the bottom of the bottom flange. It is expected that the relationships between the strain gages on opposite side of the continuity joint along with the relationship between the strain gage at the center of the span and those at the ends will provide adequate data to determine the degree of continuity. The strain gage at the center will also help in determining that there is loading on the bridge since it was not possible to apply controlled loading. Figure II.7 shows a plot of strain gage readings at the bottom of the bottom flange at the ends (B10-End and B16-End) and at the bottom of the bottom flange at the center (B16-Center). Note that strains at the level of the top flange at the ends (total of four) were very small and for clarity they are not shown here. It appears that under service load the neutral axis is near the top girder flange.

Figure II.7 Strain gage measurements (I-287 Over Darlington Ave.)

Figure II.7 indicates that full continuity does exist at this connection even under service load. Note that effort was made to ensure that scanning of strain gages are made only when a truck (of unknown weight) crosses the bridge. The peaks depict the times when the truck is in the middle of the B10 girder (around 0.8 sec or 40*0.02) and when it is in the middle of B16 girder (around 2 sec or 100*0.02). The time difference between the two peaks is the time for the truck to travel 120-ft. Comparing B10-End curve to B16-End curve (i.e., strains at opposite sides of the continuity connection) one finds that they follow the same pattern and that they are, more or less, of equal magnitude. This indicates that a good degree of continuity exists at the joint. Furthermore, the relative values of the end strains when the truck is at the center of each span are consistent (around -8 microstrains for both t = 0.8 sec and 2 sec). A picture of the continuity connection on the outside girder line, shown in Figure II.8, also does not show any visible cracks further supporting the fact that there might be very good degree of continuity. The relationship between center strain (B16-Center) and end strains is consistent and similar to that of a two span continuous beam with single intermediate support and a concentrated load at mid-span. However, their relative
values (ratio of 3 to 4) are not in agreement with that for a continuous beam (with a ratio of 13/6 or 2.1). Of course such comparison could be misleading due to differences in boundary condition and loading (single concentrated load as oppose to 3-axel truck load). These will be addressed using finite element method. The relative value of the peaks of the B16-Center curve is also not consistent with the response of a continuous beam. The ratio of the strain at B16-Center when the load is at the center of B16 girder (i.e., load on the strain gage, t = 2 sec) to the strain at the same location when the load is at the center of the B10 girder (t = 0.8 sec) is more than 6 (one can argue as high as 7.5). For a two span continuous beam this ratio is 4.3 (or more exactly 13/3). This probably indicates that the joint is not fully continuous but there is a good degree of continuity. Work is needed to come up with a common definition for “degree of continuity,” and this will be one objective of the analytical task under this study. However, based on the relative values of strains at the center compared to that at the end of the girder for this bridge versus that for a two span continuous beam the continuity index would be 0.6 to 0.5 (i.e., 2.1/3.5 to 2.1 /4). As discussed before because of the presence of anchor bolts and the fact that the pier support consists of two bearing pads of some length as oppose to a knife edged single support assumed for continuous beams such a comparison is not appropriate. Dynamic effects and nature of load application can further compound such a comparison. These data along with data from other instrumented bridges will be studied in detail to better understand the response of these connections and to determine the level of continuity. As part of this effort 2-D (and possibly 3-D) finite element models of the connection will also be employed to further enhance our knowledge of the actual behavior of these connections.

**Figure II.8** Continuity connection on outside girder line (I-287N Over Darlington Ave.)
I-287 Over Skyline Drive: This is a two span bridge with seat abutments and continuity connection at the center pier. As shown in Figure II.9 the diaphragm at the continuity joint is cracked right through the middle. This is consistent with what is typically expected to happen when time-dependent positive moment exceeds the cracking moment of the cross section. Due to access limitations it was not possible to install a strain gage at the center of any one of the two spans. For the same reason it was not possible to instrument an internal girder and the shoulder girder was instrumented. It should be added that there was an entrance ramp at this level, further distancing the shoulder girder from the center of the roadway where there was heavy traffic. Thus, instrumentation included only strain gages at the end of the girders on both sides of the joint. Similar to the previous bridge a total of three gages were installed per each end section. That is, one strain gage was installed on the bottom of the bottom flange and two were attached to the opposite sides of the top girder flange (under the deck). Despite several attempts no significant output was measured in any one of the strain gages. This could be either due to a lack of any continuity (i.e., no bending strain at the end of a simply supported beam) or it could be due to the fact that the shoulder girder does not carry much, if any, of the roadway load. Additional measurements were made while driving an NJDOT truck over the shoulder of the ramp (right over the girder instrumented) and still there were no readings. This combined with visibly wide crack at the joint indicate that this connection provides zero continuity under live load.

Figure II.9 Diaphragm crack at continuity joint (I-287 Over Skyline Dr.)
Route 1 and 9 SB Viaduct (Section 2AG): This is a long multi-span bridge where most spans are precast prestressed concrete girders with many continuity joints at the piers. It is a relatively new bridge that developed early cracking in the continuity joints. A report by Hardesty and Hanover indicates that the first inspection cycle on 3/1/99 “revealed fine to 1/16” wide cracks in the continuity diaphragms over Piers 14, 15, 18, 19, 20, 23, 26, 27, 32, 35 & 38 (11 total).” A follow up inspection on 5/5/00, where only readily accessible piers were inspected, revealed additional cracks in diaphragms over Piers 6, 8, 10, 11, and 12. In their May 18, 2000 memo, Hardesty and Hanover recommend to re-rate “Span 21 as simply supported beam for all load cases.”

Figures II.10 and II.11 show damage to two continuity joints on this bridge. While the cracking of the diaphragm in Figure II.10 is extensive and quite wide, it is consistent with what is normally expected. That is, similar to I-287 Over Skyline Dr. Bridge a vertical crack develops at the center of the diaphragm and propagates upward. However, the continuity joint shown in Figure II.11 has essentially disintegrated (the diaphragm appears to be crashed). In addition to quality of construction and material used, it is suspected that the design of the joint may also play a role. As it was discussed before, detail of continuity joint per NJDOT design manual specifies only one 1” diameter anchor bolt per line of girder. Review of plans for the previous two bridges indicated that this guideline was followed. However, in this bridge the plans indicate that four 2.5” diameter bolts are used per line of girder. The use of larger and higher number of bolts will result in development of a significantly larger pull out force that can damage/break the diaphragm. As it was discussed, the pull out force develops because the anchor bolts tend to resist rotation (see Figure II.4).

Another interesting observation from this bridge is the fact that there appears to be even damage to the girders probably due to rotational rigidity of the joint. Figure II.12 shows a close up of the bearing pad at another continuity joint. As it was mentioned before, in order to behave like continuous beams it is important that the continuity connection does not restrain rotation. This was the reason for recommending elimination of the anchor bolt(s) and debonding the girders from the diaphragms. It should be also mentioned that in addition to anchor bolts and bonding of the girders, other factors such as geometry and material characteristics of the bearing pads complicate the response of the connections. The behavior of a connection consisting of two wide bearing areas with anchor bolts in between and girders embedded in the diaphragm concrete is certainly different than a single knife edged support that is assumed in analysis of continuous beams. The finite element method provides a powerful tool to evaluate the response of such complicated joint under both gravity and time dependent loads. Development of the finite element model along with some preliminary results is briefly discussed in the next section.

This bridge is also selected for field measurement. Two continuity joints have been instrumented and currently data are being collected. Results will be discussed during future quarterly meetings. The entire data for all bridges will be presented in the final report.
Figure II.10 An internal continuity joint (Route 1 & 9 SB Viaduct).

Figure II.11 An external continuity joint - Route 1 & 9 SB Viaduct (left).

Figure II.12 Girder damage above bearing pad at an external continuity joint - Route 1 & 9 SB Viaduct (above).
Finite Element Modeling and Analysis

The purpose of the finite element analysis (FEA) part of this study is multifaceted and it has included development of both 2-D and 3-D models. Among the objectives of the FEA is evaluation of connection performance considering various factors such as girder age at the time of deck and diaphragm casting, girder age at the time of live load application, construction sequence, creep and shrinkage properties, patterned loading, level and nature of loading, and span length. ANSYS with modified creep and shrinkage models is employed to develop a 3-D model for this part of the analytical study. In the 3-D model solid elements (Solid65) are used to model girder, deck and diaphragm concrete. The cross section and an isometric view of a typical model are shown in Figure II.13. Note that due to symmetry only half of the cross-section is modeled. Nonlinear concrete material model with cracking and crushing capability is employed. For a better representation of creep and shrinkage, ACI Committee 209 models (9) are implemented into ANSYS as a part of this study and are used in the nonlinear analysis. Reinforcement and prestressing strands are modeled with 3-D bar elements with a bilinear material model. The PCI strand relaxation model (10) is also implemented into ANSYS and is used with those elements modeling prestressing strands to estimate prestress losses. The element birth and death options are used to model staged construction and development of continuity. As part of the verification of the FE model, the PCA’s half scale specimens were modeled and the results were compared with both experimental results and those reported by Mirmiran et al. (7). Results were in good agreement, however, for brevity they are not presented here. These and other verification examples will be included in the final report.

Figure II.13 3-D finite element model of typical girder and deck.
The model is now being used for parametric study. An example of the results is shown in Figure II.14 where the effect of girder age when continuity is established is investigated. The model is similar to the two-span continuous portion of I-287N Over Darlington Avenue. Only one girder and its tributary deck are modeled. The girders are AASHTO Type VI girders. The deck is 7.5-inch thick. Each span is 120-ft long. Positive reinforcement at the joint (based on the plans) consists of 8 prestress strands extended from girder ends. This provides a positive moment capacity equal to $2.4M_{cr}$ where $M_{cr}$ is the cracking moment of the composite section. Ultimate creep coefficient for both girder and deck concrete is assumed to be 3.25, and ultimate shrinkage is equal to $600 \times 10^{-3}$ in./in. Both of these values are selected based on NCHRP Report 322 and produce high creep and low shrinkage in order to highlight effect on positive moment and joint cracking. Girder age at deck casting is chosen to be 10, 28, 60 and 90 days, respectively. Live load is applied 2 years after continuity is established. Figure II.14 shows development of restraint moment at the center support. For 10-day old girder, little differential shrinkage develops and creep dominates. Thus, the end moment is positive from the start and increases with time. As moment increases the diaphragm concrete cracks subsequently the reinforcement steel yields making convergence of the solution difficult at time of 138 days. Creep also dominates the 28-day case. For 60-day and 90-day girder age cases, differential shrinkage effect is significant, thus, initially negative moment develops at the joint. It then becomes positive as creep strain accumulates. Upon application of the live load the moment at the joint become negative. Regardless of girder age and amount of cracking, under ultimate load all the cracks will close and the joint will become fully continuous.

![Figure II.14 Restraint moment at the center (pier) support: Effect of girder age](image-url)
As it can be seen from Figure II.14, girder age has significant effect on restraint positive moment at the continuity joint and the potential for cracking, which in turn affect the degree of continuity under service load. This is the reason for TNDOT specifying a minimum age of 90-day for the girders. However, it should be noted that older girders could have an adverse effect on transverse deck cracking, which itself is an issue and is the subject of another study by the PI (12). Effect of girder age on deck cracking can also be observed from Figure II.14 for the older girder cases as the slope at the end tail of the curves changes due to cracking of the deck. Therefore, at this time no recommendation is made with regard to girder age. Another parameter considered is the amount of positive reinforcements, which is shown not to have much effect, if any, especially on mid-span moment under service load. This is the reason for NCHRP Report 322 recommendation to eliminate positive reinforcement. This will be further evaluated and a recommendation will be made with the final report.

The finite element work also includes dynamic analyses of two span beams subjected to moving axel load to better understand and assess the experimental results. As it was mentioned before, the nature of loading under typical truck load (3-axel and dynamic) is complicated and not quite similar to that of a continuous beam subjected to a concentrated load at mid-span. Finite element method provides a powerful tool to perform such analysis. In the FE model, one girder and tributary deck are modeled using beam elements and 3-axel truck loading is represented with three concentrated loads. Using the arrival time option for the loading function the effect of a truck traversing the bridge is simulated as the three-concentrated loads arrive at various nodes of the model at different time depending on truck speed being simulated. The PI has successfully performed this approach in a previous study (11). Figure II.15 shows the time histories of moments at the center support and at mid-span of a two span bridge crossed by a HS20 AASHTO load (8k-32k-32k that are 14-ft and 30-ft apart) at the speed of about 70 mph (1,232 in/sec). The beam properties and length are similar to the right two spans of I-287N over Darlington Avenue. Thus, with a total length of 240-ft it takes about 2.35 second for the truck to cross the bridge and the last axel will clear the bridge in 2.8 seconds. The mid-span moment corresponds to the center of the right span. As it can be seen from this figure the ratio of the maximum moment at the center of the span to that at the pier support is less than 2 (equal to 1.7). As it was discussed before, this ratio for the same system under a concentrated static load is 2.1 and for the experimental results it is in the range of 3 to 4. Although dynamic effects were considered, in this case they did not have much effect on the maximum response. The fundamental frequency for the system is about 3 Hz.

Further finite element analyses will be conducted to gain better knowledge of the actual response and to evaluate the experimental results. In addition to the above model, 2-D solid elements will also be employed to develop finite element models that can represent more accurately the details of the continuity joint including the anchor bolt(s) and bearing pads. Results of such analyses will be used to develop guideline on analysis of this type of bridge as oppose to a continuous bridge on single knife edged support.
Future Plan

Consistent with the original research plan the following tasks will be conducted during the remaining project time:

- Continuation of finite element analysis using 2-D and 3-D models. In addition to parametric study and evaluation of in-field measurements as outlined in the original proposal and briefly discussed here, emphasis will also be placed on understanding the load transfer mechanism and stress distributions through the existing connection with and without anchor bolt(s). Based on these results, appropriate and simple analysis approach will be proposed and if necessary further refinements to existing design will be made. Simple yet detailed analyses of the connection will also provide the knowledge base to provide a concise definition of “degree of continuity.”
- Development of new connection.
- Fabrication of specimens based on new design and laboratory tests.
- Development of new connection details, specifications, drawings, and design examples.
- Final report.

Figure II.15 Time histories of moments at the center-support and mid-span as a truck traverses the bridge.
References