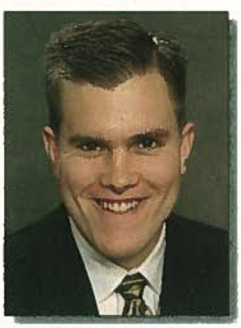
Resolving Restraint Moments: Designing for Continuity in Precast Prestressed Concrete Girder Bridges



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This paper presents analytical studies of Washington State Department of Transportation (WSDOT) standard precast, prestressed concrete bridge girders and their design for continuity. These studies indicate that deeper girders with longer spans, such as the new W83G girder, do not develop large positive restraint moments from creep and shrinkage effects. Without these large positive restraint moments, it is possible to design the girders for full or near-full continuity for superimposed loads. This continuity reduces the moment induced in each girder and leads to significant economic advantages. This paper also introduces the computer program RMCalc, developed by the authors, which simplifies the calculation of restraint moments. With the aid of this program and the potential for low restraint moments, engineers can design prestressed concrete girder spans for continuity, thereby providing greater structural efficiency in the bridge and significant cost savings to the taxpaying public.

In the state of Washington and other regions of the United States, bridge engineers often neglect continuity for positive moments while designing precast, prestressed concrete girders, even on multi-span bridges that are constructed without joints. Even though superimposed dead and live loads will act as continuous effects on the bridges, these loads typically are considered to act on simple spans with higher midspan positive moments.

The reason this practice exists is that the degree of continuity between adjacent spans for a continuous-span precast, prestressed concrete girder bridge will vary, depending on many factors related to the design and construction. Instead of performing detailed calculations to utilize what is expected to be a low percentage of continuity, engineers typically simplify the process by conservatively designing for simple spans for all loads.

Analytical studies of Washington State Department of Transportation (WSDOT) standard precast/prestressed girders have indicated, however, that it is possible to achieve significant continuity for superimposed loads on precast, prestressed concrete girder bridges, particularly for longer span bridges. A bridge that is designed for continuity will have longer span lengths or fewer lines of girders, resulting in lower overall costs compared to a simple-span design. Additionally, modern computer-based methods for computing restraint moments allow the engineer to consider continuity without having to perform extensive hand calculations.

The objective of this paper is to encourage engineers to design for continuity for superimposed loads on precast/prestressed concrete girder bridges, and to demonstrate those occurrences where continuity design is most advantageous. To demonstrate the maximum possible economic advantages of continuity, a set of design examples will compare simple spans with continuous spans that neglect the effect of restraint moments.

A second set of design examples that includes calculations of restraint moments will show the effect of restraint moments on continuity. It will be demonstrated that under certain conditions, the effect of restraint moments can be small, and, therefore, significant economic savings due to continuity design can be achieved.

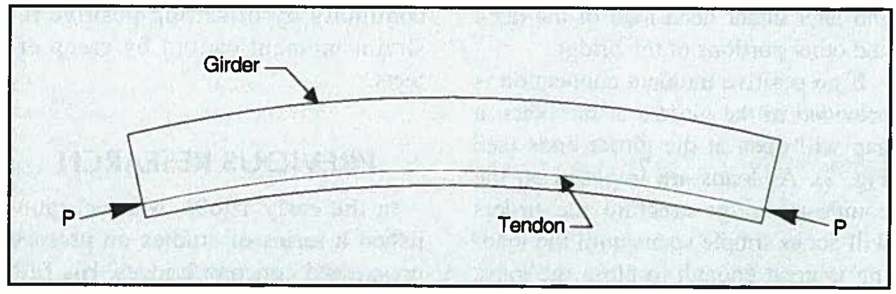

Fig. 1. Upward girder deflection due to creep.

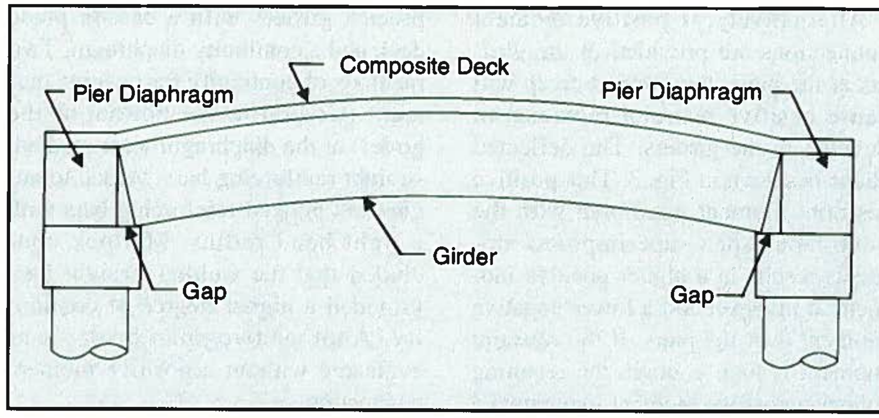

Fig. 2. Prestressed concrete girder creep without positive moment connection at piers.

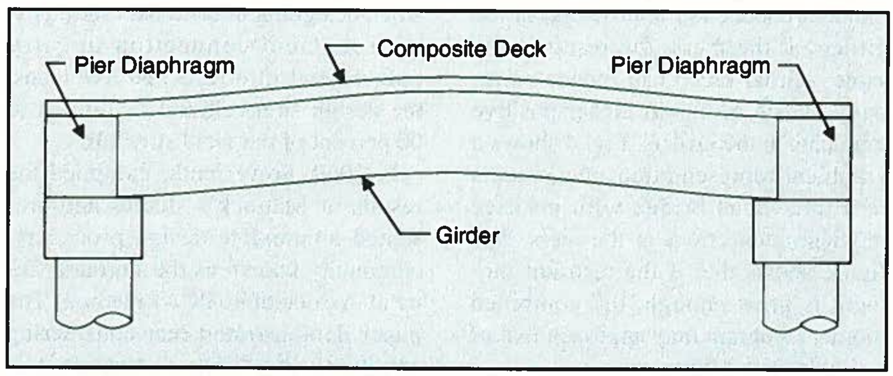

Fig. 3. Prestressed concrete girder creep with positive moment connection at piers.

EFFECT OF CREEP AND SHRINKAGE ON CONTINUITY

In general, bridge engineers can choose between two different methods for evaluating the loads transferred from the bridge girder section to the girder-slab composite section. Conservatively, the bridge can be designed such that all loads are applied to a simple-span system, neglecting time-dependent effects; this is typically done in Washington state.

Alternatively, the bridge can be designed as a continuous-span bridge for all loads applied to the composite section, adding the time-dependent effects of creep and shrinkage. This second method leads to lower construction costs and a more efficient structure.

Typically, continuous-span bridges provide negative moment continuity through tension reinforcement in the cast-in-place deck slab over the piers and through compression in the pier diaphragm. The girders act as simple spans for dead loads before the deck is made continuous over the piers. After continuity is achieved with the bridge deck, the composite section of girder and deck slab carries the superimposed dead and live loads, and the bridge behaves as a continuous structure.

Girder Creep

Over time, the continuity of a continuous bridge may be reduced by the effects of creep in the girders. The girder concrete creeps under the prestressing force, causing the girder to deflect upward (see Fig. 1). This movement is counteracted somewhat by creep under dead load of the girder and later under dead load of the deck and other portions of the bridge.

If no positive moment connection is provided in the girders at the piers, a gap will open at the girder ends (see Fig. 2). As loads are imposed on the composite bridge structure, the girders will act as simple spans until the loading is great enough to close the gaps. If the gaps are large enough, all continuity of the bridge may be lost as the girder ends hinge under applied loads.

Alternatively, if positive moment connections are provided in the girders at the piers, the upward creep will cause positive restraint moments to develop in the girders. The deflected shape is shown in Fig. 3. This positive restraint moment combined with the continuous span superimposed moments results in a higher positive moment at midspan and a lower negative moment over the piers. If the restraint moment is high enough, the resulting midspan positive moment may equal a simple-span moment, resulting in a loss of all continuity.

Regardless of whether a positive moment connection is provided in the girders at the piers, the result is the same—girder creep can reduce continuity, which results in higher positive moments in the girders. Fig. 4 shows a graphical representation of moments in a three-span bridge with positive moment connections at the piers. The figure shows that if the restraint moment is great enough, the combined moment diagram may approach that of a simple-span bridge.

Differential Shrinkage

Fortunately, creep effects are offset by differential shrinkage between the girder concrete and deck concrete. The deck concrete is younger than the girder concrete. When the deck is poured, the girders have already had time to shrink, whereas the deck concrete's shrinkage has yet to occur.

With the shrinkage of the deck concrete exceeding that remaining in the girder concrete, the result is a downward deflection of the composite deck-girder system. On a continuous bridge with negative moment connections, this results in a negative restraint moment, helping to maintain continuity by offsetting positive restraint moment caused by creep effects.

PREVIOUS RESEARCH

In the early 1960s, Mattock published a series of studies on precast/prestressed concrete bridges. His fifth study in the series, "Creep and Shrinkage Studies,"[1] consisted of analysis and half-size testing of two spans of precast girders with a cast-in-place deck and a continuity diaphragm. Two methods of continuity for positive moment (tension in the bottom of the girder) at the diaphragm were studied: straight reinforcing bars welded to angles and hooked reinforcing bars with a tight bend radius. Mattock concluded that the welded straight bars provided a higher degree of continuity. Another two-span bridge was evaluated without a positive moment connection.

The study concluded with a method to design for the effects of creep and shrinkage. It also recommended that when designing hooked bars for a positive moment connection to carry time-dependent effects and live loads, the design stress should be limited to 60 percent of the yield strength.

In 1969, Freyermuth[2] compiled the results of Mattock's studies and presented a complete design procedure, commonly known as the Portland Cement Association (PCA) method. The paper demonstrated that considering continuity in a bridge constructed of precast, prestressed single-span units required inclusion of the effects of creep and shrinkage. The complete design example in Freyermuth's paper illustrates necessary design procedures for continuity considerations.

The example includes the effects of creep and shrinkage with the development of positive restraint moments at the intermediate pier of a two-span bridge. The discussion includes the details of the positive moment connection and recommendations for the design of the connection. The design of negative moment reinforcement over the piers is also addressed.

In 1989, Oesterle et al. published NCHRP Report 322.[3] This study was purely analytical, and a revised analysis method was developed to predict time-dependent restraint moments. This method, based on the PCA method, had several improvements, including a time-step analysis, and is commonly known as the Construction Technology Laboratories (CTL) method. The computer program BRIDGERM was developed to calculate restraint moments by the CTL method.

The study concluded that positive moment connections between the ends of adjacent girders in the closure diaphragms are difficult, time consuming, and costly to install, and, furthermore, the connections add no structural benefit.

In 1998, Peterman and Ramirez[4] investigated restraint moments on bridges with full-span prestressed concrete form panels. They proposed a modification to the restraint moment calculations by the PCA and CTL methods. Their modified method, referred to as the P-method, resulted in better correlation of results for the two full-scale bridges tested in the study. Because the method is intended for bridges with precast/prestressed concrete form panels, it is not considered in the study presented here. However, the paper presents a good overview of the various methods for calculating restraint moments in any precast/prestressed bridge type.

In 2001, Mirmiran et al.[5] proposed another method for calculating restraint moments in precast, prestressed concrete girder bridges. Their method involves considering the properties of a bridge as nonlinear along a bridge's length, due to varying amounts of reinforcement and cracking. To account for these nonlinearities, the method incorporates a moment-curvature analysis at each time step.

Analytical results of the nonlinear method were compared to results from previous physical testing performed by Mattock[1] and results from the CTL method. The results indicate that this nonlinear method can achieve a higher degree of accuracy than the CTL method. However, because the method is very new, more complex, and part of an unfinished study (as of this writing) it is not considered in the present study.

CALCULATION OF RESTRAINT MOMENTS

The restraint moment in a girder consists of the summation of the girder creep restraint moment and the differential shrinkage restraint moment. Because the previously mentioned studies have already presented in detail the theory behind the calculation of restraint moments, only a general overview will be presented here.

The two primary methods used for computing creep and shrinkage restraint moments are the PCA method and the CTL method. The CTL method, while purported to be more accurate than the PCA method, is more complex and not suited for hand calculations. Therefore, the simpler PCA method is presented below for general understanding.

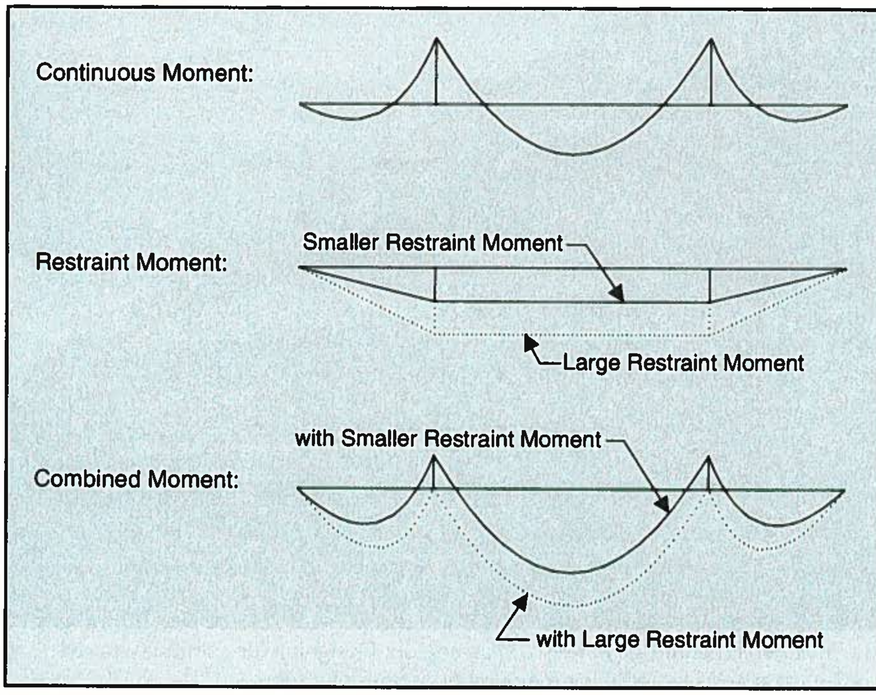

Fig. 4. Summation of moments on a three-span bridge.

Girder Creep

The calculation of creep in a precast/prestressed concrete girder requires the following parameters:

- Summation of the moments at piers due to prestressing force and dead load, applied to the continuous-span bridge (moments that are applied prior to continuity, and then locked in place by continuity diaphragms)
- Elastic modulus of the girder
- Time that pretensioning strands are released onto the girder
- Volume-to-surface ratio of the girder
- Ultimate creep coefficient based on a 20-year loading curve

The value for the ultimate creep coefficient typically ranges from 1.5 to 2.5 for prestressed concrete girder sections. The ACI report "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures," ACI 209R-92,[6] can be used to predict the ultimate creep coefficient in lieu of testing data. Calculation of the creep moment by the PCA method is reflected in Eq. (1):

$$M_c = M_{PS/DL\text{-}CONT}(1 - e^{-\phi}) \qquad (1)$$

where

$M_{PS/DL\text{-}CONT}$ = sum of moments at piers due to prestressing force and dead load (simple-span loads) applied to the continuous-span bridge

$\phi = \varepsilon_{creep}/\varepsilon_{elastic}$ = ultimate creep coefficient based on 20-year loading

ε_{creep} = strain due to creep

$\varepsilon_{elastic}$ = elastic strain due to applied loads

Freyermuth[2] provides charts and tables to assist the designer in determining all of the variables needed for computing creep moment.

Differential Shrinkage

The moments due to differential shrinkage are based on the following factors:

- Elastic modulus of the deck slab
- Cross-sectional area of the deck slab
- Distance between the mid-depth of the slab and the composite centroid of the built-up girder
- Ultimate creep coefficient based on a 20-year loading curve

The magnitude of the shrinkage moment by the PCA method is calculated by Eq. (2):

$$M_s = \varepsilon_s E_b A_b d\left(\frac{1 - e^{-\phi}}{\phi}\right) \qquad (2)$$

where

ε_s = differential shrinkage strain

E_b = elastic modulus of girder

A_b = cross-sectional area of girder

d = distance between mid-depth of slab and centroid of composite section

ϕ = ultimate creep coefficient based on 20-year loading

An additional factor related to differential shrinkage is the restraining effect of the reinforcement in the deck. Large amounts of deck reinforcement can reduce the amount of deck shrinkage. This phenomenon is known as the Dischinger effect,[3] and is incorporated into the CTL method.

Construction Timing

Several factors influence the total restraint moment in a bridge and the corresponding degree of continuity. These factors include girder age at continuity, girder geometry, prestressing strand layout, girder and deck concrete properties, and bridge geometry.

One factor over which a designer has little control is the girder age at continuity, which has a great effect on the restraint moments. The age of the girders when the bridge is made continuous determines how much girder creep and shrinkage have already occurred in an unrestrained state, and how much remains after continuity in a restrained state. The girder age de-

Fig. 5. Allen Street Bridge in Kelso, Washington. Designing for continuity saved construction costs by reducing the number of lines of girders.
Photo courtesy: © Kevin Hinkley.

pends on precasting plant production schedules, the size of the bridge and resulting construction schedule, and the timing between placing the deck over the span and placing the deck over the piers.

In general, the older a girder is before continuity is established, the lower the positive restraint moments will be, because less creep and less shrinkage remain to develop in the girder. Less remaining creep results in lower positive restraint moments due to creep. Less remaining girder shrinkage results in larger differential shrinkage between the deck concrete and girder concrete, which translates to larger negative restraint moments due to shrinkage. The combined effect is a lower positive restraint moment (less positive or more negative), which leads to greater continuity.

Many engineers may feel unable to predict construction timing accurately. In fact, NCHRP Report 322[3] states that according to a survey, the majority of girders are between 10 and 90 days old at the time of construction, a wide range. However, consideration of the size of the project, anticipated construction delays, and previous construction data can provide engineers a means to predict construction timing and enable them to make reasonable assumptions. Additionally, engineers have the option to place time limitations in project specifications.

One source of previous construction data is the Allen Street Bridge in Kelso, Washington (see Fig. 5). Designed by the authors and their associates, the bridge was completed in January 2001. It is 1114 ft (339.5 m) long and consists of seven spans with a typical configuration of eight girders spaced at 8 ft 6 in. (2.59 m) on center. Three piers in the river are spaced at 167 ft 6 in. (51.05 m) with the remaining piers spaced at 153 ft (46.63 m). The bridge uses 83 in. (2100 mm) deep W83G girders for all seven spans. (More information on the Allen Street Bridge can be obtained from a recent article in ASCENT magazine.[7])

Because this bridge was very large, the time from erection of the girders to placement of the bridge deck allowed more time for the girders to age. Thus, the girders were relatively old at the time continuity was established between spans. Because of the large size of the bridge, there were more girders to produce, transport, and erect; more deck falsework to construct; more deck reinforcement to place and tie; and more deck concrete to place before the spans were made continuous.

The youngest girder age when two spans were made continuous on the Allen Street Bridge was 91 days, which corresponds to the oldest girder age at continuity reported by the NCHRP 322 survey.[3] The oldest girder age in the first stage of construction was 144 days at continuity, and the second stage of construction had girder ages at continuity ranging from 270 to 277 days.

Even though the designer has limited control over girder age, conservative assumptions can be made, and, if necessary, reasonable limitations on girder age can be included in the project specifications. In this manner, the designer can arrive at reasonable values for restraint moments and degree of continuity.

Concrete Properties

The ultimate creep coefficient and ultimate shrinkage coefficient of the girder and deck concrete are directly related to the restraint moments developed in a girder. Typically, these coefficients are based on a 20-year loading period. These coefficients depend on the concrete composition, girder and deck geometry, and ambient relative humidity during the life of the girder. Ideally, the concrete mix designs should be tested to determine ultimate creep coefficients and ultimate shrinkage coefficients, although this is rarely done for typical girder bridges.

In lieu of testing, estimated values for the ultimate coefficients can be obtained from ACI 209R-92.[6] Factors that influence the calculations for the ultimate creep and shrinkage coefficients include ambient relative humidity, volume-to-surface ratio of the member, concrete slump, cement content, fine aggregate content, and air content.

It is expected that values for the ultimate creep and shrinkage coefficients will vary with geographic location because of regional variations in mix designs, aggregates, standard girder shapes, and weather. For the studies presented here, data for Western Washington State and the Allen Street Bridge are used to compute ultimate creep and shrinkage coefficients in accordance with ACI 209R-92.

The calculations result in ultimate creep coefficients that vary between 1.52 and 1.57 and ultimate shrinkage coefficients that vary between 282 x 10^{-6} and 292 x 10^{-6} in./in.

Calculation Methods

The actual calculation of restraint moments can be carried out through several means. The first option is hand calculation, typically using the PCA method. Usually only the ultimate restraint moments are computed using hand methods. Several of the previous studies have presented design examples using the PCA method.

The second option is to use the computer program BRIDGERM, presented in NCHRP Report 322.[3] This computer program executes the CTL method, which is an incremental time-step solution using time-dependent material properties according to ACI 209. This program provides the ability to look at the complete time-history of restraint moments. BRIDGERM was written in Fortran, and a version compiled for DOS is available from the Center for Microcomputers in Transportation at the University of Florida, as well as PC-TRANS at Kansas University.

A third option is to use the new computer program RMCalc, which was developed by the authors to compute restraint moments using the Microsoft Windows platform. RMCalc was developed using Microsoft Visual Basic, and uses the same algorithms as BRIDGERM, and thus follows the CTL method. RMCalc is essentially a repackaging of BRIDGERM and, therefore, requires the same input and produces identical results. However, RMCalc is much easier to use. RMCalc is available free of charge through the WSDOT Bridge and Structures Office's Alternate Route Project.

RMCalc is released under an open source license. This means that the engineering community is able to freely access the source code and modify or improve the program. It can be downloaded from the Internet at www.wsdot.wa.gov/eesc/bridge/software/.

In addition to the restraint moment calculator, RMCalc includes a Microsoft Excel spreadsheet to assist the engineer in determining the input criteria. The spreadsheet computes ultimate creep and shrinkage coefficients according to ACI 209R-92. Analyses for this paper were performed using RMCalc and Excel.

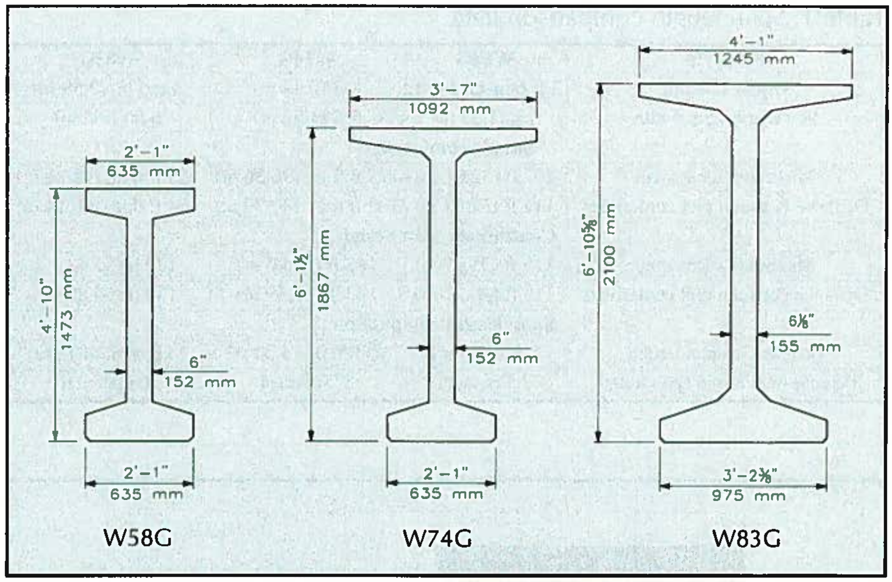

Fig. 6. WSDOT standard girder cross sections used for analysis.

ANALYTICAL STUDIES

The following analytical studies will demonstrate that it is advantageous to design for continuity and consider restraint moments for some bridges. In particular, it will be demonstrated that larger girders and longer span bridges have lower positive restraint moments and, therefore, higher resulting degrees of continuity.

No physical testing has been performed for this paper. Instead, restraint moments were analyzed using the CTL method with the software program RMCalc.

Design examples will be provided using three standard WSDOT girder types: W58G, W74G, and W83G. The numbers in the girder designations are the approximate girder height in inches (see Fig. 6).

The development of the W83G, originally known as the W21MG, was led by Seguirant,[8] who introduced two new girders in 1998. A larger version, the W95G, was also developed and adopted by the WSDOT; however, its weight makes it better suited to segmental post-tensioned applications due to transport weight limitations.

Each design example considers a single interior girder in a five-span bridge with eight girders per span. The designed girder is located in the center span, carries a portion of the curb load, and is fully pretensioned. This configuration was chosen to avoid influences due to short end spans and deck overhangs.

Girder concrete release strength is limited to 7.0 ksi (48 MPa) to permit a one-day turnaround on girder production. (Higher release strengths would require the girders to remain in the precasting bed longer, raising production cost.) The 28-day girder concrete strength was limited to 8.0 ksi (55 MPa), although the release strength typically controlled the designs. All prestressing strands are 0.6 in. (15 mm) diameter low-relaxation strands.

Temporary prestressing strands debonded in the center portion of the girder were used in the top flanges of the girders to reduce the required release strength and improve stability for transport. These strands are cut after the girders have been erected and before the deck is cast. Four temporary prestressing strands are used in the W58G girders, and six are used in the W74G and W83G girders.

All bridges have a 7.5 in. (190 mm) thick cast-in-place concrete deck. The deck concrete has a 28-day strength of 4.0 ksi (28 MPa), based on the WSDOT Class 4000D standard deck concrete mix.[9] Dead loads are the girder and deck weights of 160 pcf (2563 kg/m^3), with a tributary traffic

Table 1. Span length comparison data.

Property	W58G	W74G	W83G
Girder spacing	7 ft 6 in. (2.29 m)	8 ft (2.44 m)	8 ft 6 in. (2.59 m)
Pier diaphragm width	4 ft (1.22 m)	5 ft (1.52 m)	6 ft (1.83 m)
Simple span design			
Maximum clear span	107 ft (32.61 m)	127 ft 6 in. (38.86 m)	158 ft 6 in. (48.31 m)
Distance between pier centerlines	111 ft (33.83 m)	132 ft 6 in. (40.39 m)	164 ft 6 in. (50.14 m)
Continuous span design			
Maximum clear span	115 ft (35.05 m)	138 ft (42.06 m)	172 ft (52.43 m)
Distance between pier centerlines	119 ft (36.27 m)	143 ft (43.59 m)	178 ft (54.25 m)
Span length comparison			
Increase in span length	8 ft (2.44 m)	10 ft 6 in. (3.20 m)	13 ft 6 in. (4.11 m)
Percent increase in span length	7.5 percent	8.0 percent	8.5 percent

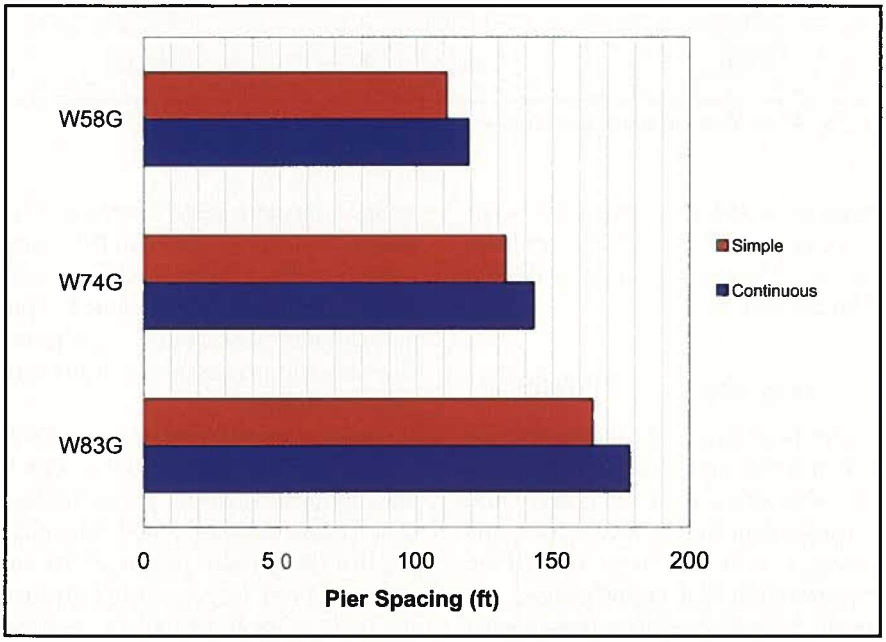

Fig. 7. Simple-span design versus continuous-span design maximum span length.

barrier load of 150 lbs per linear ft (223 kg/m) and a wearing surface of 25 psf (122 kg/m^2). To simplify the remaining loading, an additional dead load equal to 30 percent of the deck weight is assumed to account for the deck haunch and interior diaphragms. Live load is AASHTO HL-93 with applicable impact factors.

The sign convention for the design comparisons and discussions herein is based on a positive moment developing tension in the bottom of the girder, and a negative moment developing tension in the top of the girder or deck slab. Design was performed in accordance with the AASHTO LRFD Bridge Design Specifications.[10] Additionally, in accordance with WSDOT design policy, bottom tensile stress in the girders is limited to zero.[11]

Span Length Comparisons

Maximum span lengths were calculated for all three girder types. Span lengths were computed assuming full continuity for superimposed loads and also simple spans. The two resulting span lengths per girder will be compared.

For the continuous-span designs, live load moments were determined using the WSDOT's Alternate Route Project software program QConBridge. Girder designs were carried out using an in-house Microsoft Excel spreadsheet program, which has been used in the past for construction projects. Simple-span designs, including moment calculation and girder design, were carried out using the WSDOT's Alternate Route Project software program PGSuper.

For this comparison, the effect of restraint moments is not considered. The continuous designs are based on 100 percent continuity, and therefore are best-case-scenario, upper-bound solutions. The simple-span designs are based on zero percent continuity and thus are worst-case-scenario, lower-bound solutions.

The girder spacing and pier width are proportioned for each girder type. A summary of these comparisons is shown in Table 1. Fig. 7 provides a graphical comparison of span lengths.

It can be seen from Table 1 that designs based on continuity for superimposed loads have advantages over simple-span designs for all three girder types. By increasing span lengths as much as 8.5 percent, precast, prestressed girder bridges become more economical. Although it is difficult to quantify monetary savings, longer spans mean fewer piers, smaller girders for clearance and vertical curvature concerns, and/or the use of precast concrete girders in lieu of other more expensive spanning solutions.

Fewer bridge piers could also mean fewer construction seasons and thus time and monetary savings, as was the case for the Allen Street Bridge.

It should be emphasized that these are upper- and lower-bound maximum span lengths for a continuous-span design. If a precast, prestressed concrete girder bridge is constructed with continuous spans, restraint moments may develop over the life of the bridge, with a positive moment connection provided at the ends of the girders. If the restraint moments are positive, tensile stress in the bottom of the girder will increase. Alternatively, if no positive moment connections are provided at the piers, gaps may open at the ends due to upward girder creep, causing the girders to act as simple spans until the gaps close. This also results in additional tensile stress in the bottom of the girder.

Since the span lengths presented have already maximized the bottom girder stress, the presence of significant positive restraint moments, or large gaps at the girder ends, will reduce the maximum span length for the full continuity design case. The worst-case scenario for a continuous-span design is if the sum of the continuous-span moments and the restraint moments equals the simple-span moments, or, without positive moment connections at the piers, if large enough gaps open at the piers so that the girders hinge under all loads. Hence, the simple-span design span length is the lower-bound value for span length.

Girder Spacing Comparisons

Precast prestressed concrete girders become more efficient by designing for continuity for superimposed loads. Rather than using longer spans, one can achieve the benefits of continuity by using girders on shorter spans (ones that would be required for a simple-span design) with wider spacing. A wider girder spacing can sometimes eliminate one or more lines of girders, resulting in a significant cost savings.

The continuous-span designs are compared to the simple-span designs using a constant span length. For each comparison, the span length is the maximum simple-span length computed previously. It will be shown that in each case, one line of girders can be eliminated when continuity is considered, even with only partial continuity. In each case, the bottom fiber stress in the girder is determined from the SERVICE-III load case. A summary of these comparisons is shown in Table 2.

While the cost savings due to longer span lengths are probably significant, the savings from eliminating girders on the same span length are more easily identified. It is shown in Table 2 that when eliminating one girder line and designing for continuity, there is still compressive stress in the midspan bottom fiber of the girder. The girder spacing comparisons have shown that \$11,000 to nearly \$24,000 per span can be saved by designing for continuity. Fig. 8 shows a graphical summary of girder costs and savings per span. These costs do not include sales tax or engineering, so actual cost savings will be greater.

It should be noted that because current WSDOT practice is to design the girders as simple spans but to construct them as continuous with positive moment connections at the piers, there is no cost differential associated with the pier diaphragm or continuity reinforcement. However, the savings associated with a wider girder spacing may be offset somewhat by more deck reinforcement and higher forming costs.

The costs for continuous design, and the associated savings, shown in Fig. 8, are based on full continuity and therefore are upper-bound values. However, because there is reserve capacity in each girder in the continuous layout, as demonstrated by the bottom fiber compressive stress, these figures are also reasonable for girders with low restraint moments.

Table 2. Girder spacing comparison data.

Property	W58G	W74G	W83G
Deck width	60 ft (18.29 m)	64 ft (19.51 m)	68 ft (20.73 m)
Clear span	107 ft (32.61 m)	127 ft 6 in. (38.86 m)	158 ft 6 in. (48.31 m)
Simple span			
Number of girder lines	8	8	8
Girder spacing	7 ft 6 in. (2.29 m)	8 ft (2.44 m)	8 ft 6 in. (2.59 m)
Continuous span			
Number of girder lines	7	7	7
Girder spacing	8 ft 6.9 in. (2.61 m)	9 ft 1.7 in. (2.79 m)	9 ft 8.6 in. (2.96 m)
SERVICE III midspan bottom compressive stress	100 psi (4.8 kPa)	200 psi (9.6 kPa)	225 psi (10.8 kPa)
Cost comparison			
2002 WSDOT girder costs	\$105/ft (\$344/m)	\$115/ft (\$377/m)	\$150/ft (\$492/m)
Savings per span	\$11,200	\$14,600	\$23,700

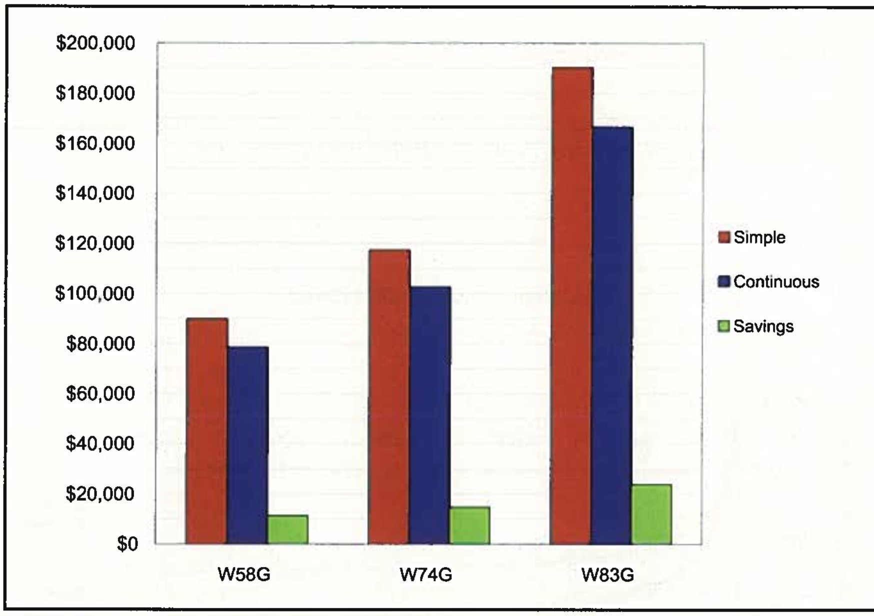

Fig. 8. Simple span versus continuous span girder costs per span.

Restraint Moment Comparisons

Restraint moments are computed for each bridge type to show how much restraint moments affect continuity. To demonstrate the impact of construction timing, restraint moments were computed for all three girder types using girder ages of 30, 60, and 90 days at the time of continuity. These girder ages were chosen assuming that 30 days was the earliest practical girder age for smaller bridges, and 90 days was the earliest practical girder age for large bridges such as the Allen Street Bridge.

For each bridge example, two effects will cause restraint moments to develop. First, a negative moment develops due to the differential shrinkage between the deck slab and the girder. The highest magnitude of this load develops shortly after continuity is established. At the same time, a positive moment begins to develop due to

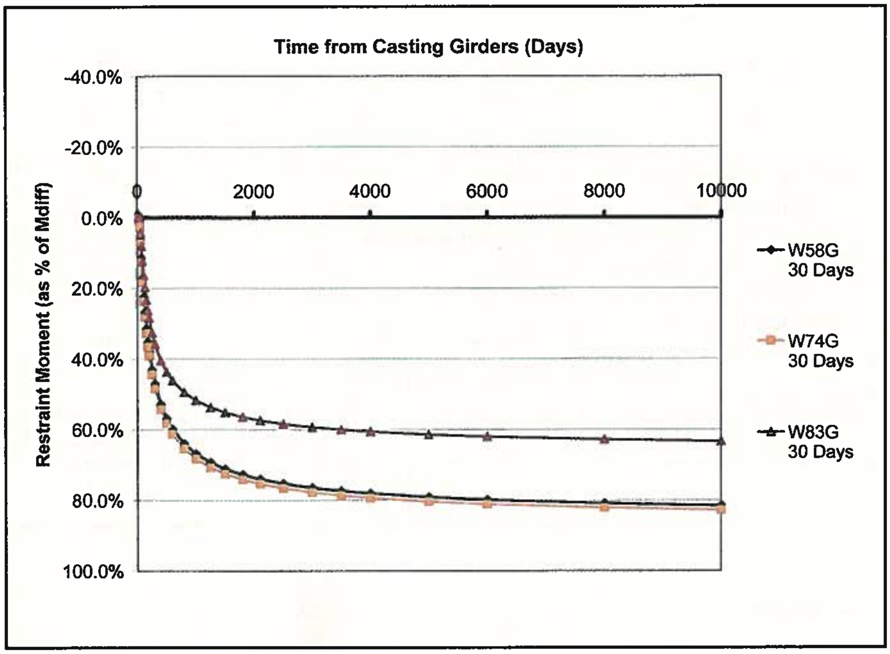

Fig. 9. Percent restraint moments at 30-day girder age at continuity.

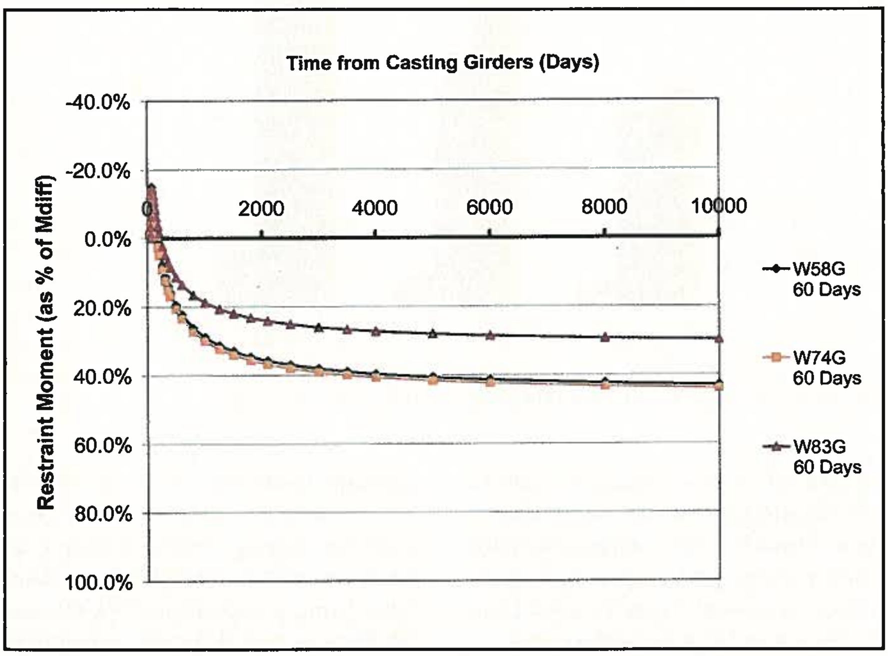

Fig. 10. Percent restraint moments at 60-day girder age at continuity.

girder creep caused by prestressing forces. The sum of the two moments is the developed restraint moment, which is almost always negative initially, but may later become positive depending on net creep effects.

Because the primary concern with restraint moments is their reduction of the effective continuity in a bridge, the restraint moments have been calculated as a percentage of the difference between the full-continuity SERVICE-III design moments and the simple span only SERVICE-III design moments at midspan (M_{diff}). Thus, a zero restraint moment indicates that full continuity is retained, whereas a 100 percent restraint moment indicates that no continuity is retained, and the structure behaves as a simple-span structure.

Note that for the W58G, W74G, and W83G girder bridges, the continuous span maximum midspan positive moments are, respectively, 19, 17, and 15 percent lower than the simple span maximum midspan positive moments. Therefore, a 50 percent restraint moment in a continuous W83G girder bridge would mean that the maximum midspan positive moment is 7.5 percent lower than the simple span maximum midspan positive moment.

In WSDOT practice, the deck concrete is poured initially on the girder spans, but not over the piers. After the first pour has cured, the remaining deck is poured over the piers. For this study, it is assumed that the first deck concrete pour occurs 14 days before the second pour when the structure is made continuous.

Because the girder ends are free to rotate when the deck is first placed, no restraint moments develop due to deck shrinkage initially, which otherwise would cause a negative restraint moment over the piers. Therefore, the tendency for deck cracking over the piers is reduced. However, this means there is less negative restraint moment to offset future positive restraint moments that will develop.

Reversing the order of deck placement (i.e., placing concrete over the piers first) would ultimately lead to lower positive restraint moments and greater continuity. However, it would require greater negative moment reinforcement to prevent negative restraint moment cracking. This scenario is not considered in this study.

30-Day Girder Age at Continuity — Fig. 9 shows a plot of restraint moments for a girder age of 30 days at the time adjacent bridge spans are made continuous. Note that there would be little advantage to designing for continuity with W58G or W74G girders that would be made continuous when the girders are only 30 days old. The W83G girder shows more promise for this situation, but still loses over 60 percent of its continuity over its life span.

60-Day Girder Age at Continuity — Fig. 10 is a plot of restraint moments for a girder age of 60 days at the time adjacent bridge spans are made continuous. The graph shows that increasing the girder age to 60 days at the time adjacent spans are made continuous decreases the positive restraint moments for the W58G and W74G girders to approximately 40 percent of the difference between the simple-span design and continuous-span design moments. The advantage is even greater for the W83G girder – less than 30 percent of continuity is reduced by restraint moments.

90-Day Girder Age at Continuity — Fig. 11 shows a plot of restraint moments for a girder age of 90 days at the time adjacent bridge spans are made continuous. The graph demonstrates that near-full continuity is possible for girder ages of 90 days at the time adjacent spans are made continuous. The W58G and W74G girders show a reduction in continuity of approximately 20 percent over their lifetimes, and the W83G girder shows approximately a 10 percent reduction in continuity. Based on the trend established by these figures, it is clear that for girder ages greater than 90 days at time of continuity, 100 percent continuity is achievable, enabling upper-bound span lengths to be used.

For bridges with less than 100 percent continuity, an engineer will need to use a span that is shorter than the upper-bound span length to allow the girders to carry the restraint moments. This results in an iterative process, since changing the span length will have an effect on the restraint moments. For bridges with restraint moments that approach 100 percent of the difference between the simple-span and continuous-span midspan moments, (i.e., zero continuity), the resulting span length will be the lower-bound, simple-span design span length.

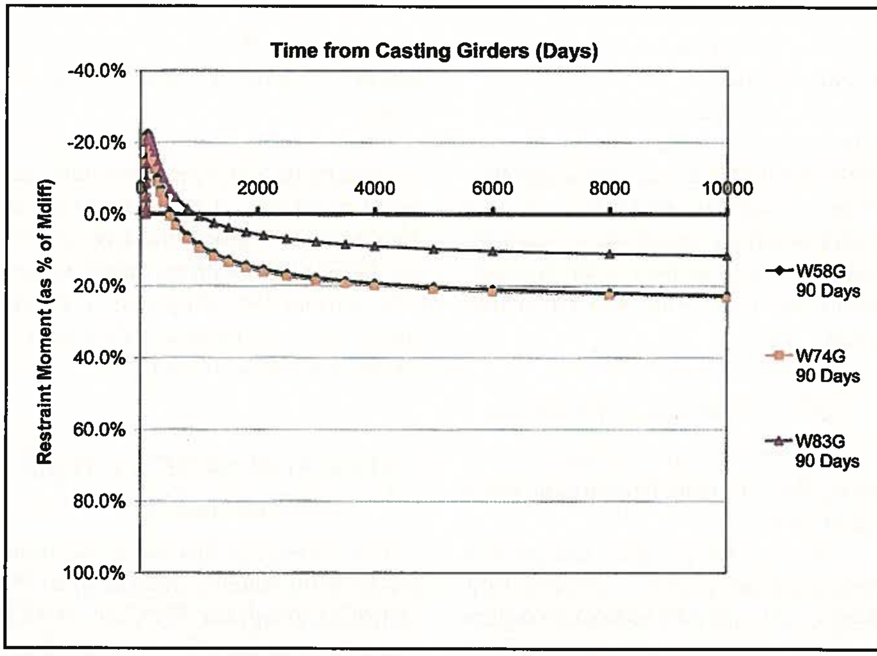

Fig. 11. Percent restraint moments at 90-day girder age at continuity.

DISCUSSION OF MAJOR FINDINGS

The previous comparisons have demonstrated several findings. First, where continuity for superimposed loads can be achieved and maintained, there is a significant economic advantage in accounting for continuity during the design process. Bridge designs based on continuity have potentially longer span lengths and wider girder spacings, compared to designs that discount continuity effects.

The increase in span length by assuming full continuity is as much as 8.5 percent for the conditions considered, which translates to a 13.5 ft (4.11 m) increase in span length for the W83G case. Alternatively, an economic advantage can be obtained by using a wider girder spacing in a continuous design compared to a simple-span design with the same span length, eliminating one or more girder lines, resulting in savings of as much as $24,000 per span.

These studies have also shown that some of the potential gains from designing for continuity may be offset by additional stress in the girders caused by restraint moments. Since it is expected that shallower girders will be used on smaller bridges (shorter spans, fewer spans, and narrower spans), these girders may be more likely to be younger when continuity is made in a bridge. The combination of shallower and younger girders leads to large restraint moments and thus significant reductions in continuity. In this situation, it may be practical to ignore continuity and restraint moments and simply design for simple-span loading.

However, the deeper WSDOT W83G girder appears to have lower restraint moments than shallower girders. Additionally, it is expected that deeper girders will be used on larger bridges (longer spans, more spans, and wider spans). A larger bridge typically means that the girders will be older when continuity is achieved in a bridge.

In the comparisons presented here, the combination of deeper and older girders leads to smaller restraint moments, possibly even no long-term positive restraint moments for older girders. Thus, near-full continuity for superimposed loads can be achieved on larger bridges constructed of precast prestressed concrete girders. This means there are substantial economic advantages to designing for continuity and considering restraint moments.

CONCLUSIONS

Based on the data presented in this paper, the following conclusions can be made:

1. Designing precast, prestressed concrete bridge girder spans made continuous for superimposed loads creates potentially significant cost savings.

2. To design for continuity, engineers must consider the effect of creep and shrinkage restraint moments.

3. The Washington State Department of Transportation's standard W83G girder shows significant potential for having low positive restraint moments, therefore maintaining significant continuity throughout a bridge's lifetime.

4. The software program RMCalc,

in conjunction with ACI 209R-92, provides a simple means of computing restraint moments.

5. The combination of recently developed deep precast prestressed concrete girder shapes and new Windows-based computing methods, makes it practical and advantageous to consider continuity in the design of precast, prestressed concrete bridge girder spans.

RECOMMENDATIONS

Based on the data presented in this paper, the following recommendations are offered:

1. During design, consider project size, location, potential construction delays, and previous construction data to estimate construction timing for precast girders.

2. For shallower girders such as the WSDOT W58G or smaller, with expected girder ages of 30 days or less at time of continuity, ignore continuity and design girders as simple spans.

3. For all other situations, consider continuity during girder design to achieve the most cost efficient structure. The design should include the effects of restraint moments, which can be easily evaluated using the computer program RMCalc.

4. For best continuity behavior, use deep girders such as the WSDOT W83G, at the girders' maximum spanning capability.

5. To achieve near full continuity, girder age at the time continuity is achieved should be approximately 90 days or older.

This paper is based on current design methods and typical concrete material properties in Washington state. Engineers in regions that have different design philosophies should undertake similar investigations to determine how applicable this paper's results are to their region.

RECOMMENDED FUTURE RESEARCH

The findings in this paper are based partially on results obtained from the software program RMCalc, which uses the same algorithms and produces the same results as the older program BRIDGERM to compute restraint moments. BRIDGERM was developed as part of NCHRP Report 322,[3] which was based solely on analytical results, using 130 ft (39.62 m) as the longest span considered. In addition to analytical correlation, NCHRP Report 322[3] compared results to physical testing performed by Mattock.[1] Mattock's testing involved two half-scale structures, built to model two-span bridges with 66 ft (20.12 m) spans.

None of the prior research involving continuity and restraint moments in precast prestressed concrete girder bridges has considered span lengths greater than 130 ft (39.62 m), and no known physical testing for restraint moments has been performed on bridges with more than two spans or with spans longer than 66 ft (20.12 m). Therefore, an opportunity for research exists to perform physical testing of long span and/or multi-span bridges to verify that the current methods for computing restraint moments are applicable to bridges with more than two spans and with span lengths greater than 130 ft (39.62 m).

High strength and high performance concrete is commonly used for pretensioned construction in Washington state and other parts of the United States. Concrete strengths of 8 ksi (55 MPa), as used in this study, and 10 ksi (69 MPa) and higher are becoming more common. Since the creep and shrinkage predictions of ACI 209 were originally published in 1982, a review of the applicability of guidelines in ACI 209R-92[6] to modern high strength concrete is needed. In addition, a review of other existing studies and methods to compute creep and shrinkage in high strength concrete would be useful, with possible additional research as needed.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the efforts of the PCI JOURNAL reviewers, whose constructive input has helped to improve the paper.

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APPENDIX A – DATA USED IN CALCULATIONS

Ultimate Creep and Shrinkage Coefficients

Calculations according to ACI 209R-92[6]

Girder Concrete

Air Content = 1.5 percent
Cement Content = 658 lb/yd^3 (390 kg/m^3)
Fine Aggregates = 31.3 percent
Slump = 5.0 in. (127 mm)
Volume to Surface Ratio = 3.1 (W58G); 2.9 (W74G); 3.2 (W83G)
Loading Age = 1 day, steam cured
Ambient Relative Humidity = 80 percent

Correction factors	γ_{creep}	$\gamma_{shrinkage}$
Air Content	1.000	0.962
Cement Content		0.947
Fine Aggregates	0.955	0.738
Slump	1.155	1.095
Loading Age	1.000	
Relative Humidity	0.734	0.600
Volume to Surface (W58G)	0.808	0.827
Volume to Surface (W74G)	0.824	0.847
Volume to Surface (W83G)	0.800	0.817

Ultimate Creep Coefficient = $2.35 \times \gamma_{creep}$
Ultimate Shrinkage Coefficient = $780 \times 10^{-6} \times \gamma_{shrinkage}$

	γ_{creep}	$\gamma_{shrinkage}$	Ult. Creep. Coeff.	Ult. Shrink. Coeff.
W58G	0.654	0.336	1.54	285.2×10^{-6}
W74G	0.667	0.375	1.57	292.1×10^{-6}
W83G	0.648	0.361	1.52	281.8×10^{-6}

Deck Concrete

Air Content = 6.0 percent
Cement Content = 660 lb/yd^3 (392 kg/m^3)
Fine Aggregates = 26.0 percent
Slump = 3.5 in. (89 mm)
Average Thickness = 7.5 in. (190 mm)
Loading Age = 14 days, moist cured
Ambient Relative Humidity = 80 percent

Correction factors	$\gamma_{shrinkage}$
Air Content	0.998
Cement Content	0.948
Fine Aggregates	0.664
Slump	1.034
Relative Humidity	0.600
Volume to Surface	0.953

Ultimate Shrinkage Coefficient = $780 \times 10^{-6} \times \gamma_{shrinkage}$

	$\gamma_{shrinkage}$	Ult. Shrink. Coeff.
Deck	0.371	289.4×10^{-6}

Girder Design Examples

For simple span cases, WSDOT Alternate Route Project software program PGSuper was used for loads and girder design. For continuous span cases, WSDOT Alternate Route Project software program QConBridge was used for loads and an in-house spreadsheet was used for girder design.

W58G Girder Designs

Girder:

A_{gir} = 604 sq in. (389 677 mm^2)
I_{gir} = 265,373 in.^{4} (1.105 x 10^{11} mm^4)
w_{gir} = 670 plf (997 kg/m)
h_{gir} = 58.00 in. (1473 mm)
y_t = 30.04 in. (763 mm)
y_b = 27.96 in. (710 mm)
b_{tf} = 25.00 in. (635 mm)
b_w = 6.00 in. (152 mm)
f'_{ci} = 7.0 ksi (48 MPa)
f'_c = 8.0 ksi (55 MPa)

Slab:

t_s = 7.50 in. (190 mm)
t_{se} = 7.00 in. (178 mm)
f'_c = 4.0 ksi (28 MPa)

Prestressing:

A_{strand} = 0.217 sq in. (140 mm^2)
F_{pu} = 270.0 ksi (1862 MPa)
F_{pi} = 202.5 ksi (1396 MPa)
F_{py} = 243.0 ksi (1675 MPa)
E_{ps} = 28,500 ksi (196 500 MPa)
Low relaxation strand

Miscellaneous:

H = 80
PPR = 1.0
$t_{transfer}$ = 1 day
w_{curb} = 150 plf (223 kg/m)
$w_{overlay}$ = 25 psf (122 kg/m^2)
$w_{concrete}$ = 160 pcf (2563 kg/m^3)
Harping point = 0.4L

W58G	Simple Span Design	Continuous Span Design (Longer Span)	Continuous Span Design (Wider Girder Spacing)
Span length	107 ft	115.0 ft	107.0 ft
Girder spacing	7.5 ft	7.5 ft	8.575 ft
Haunch	4.5 in.	—	—
Interior diaphragms	31.0 x 8.0 in. at 1/3 points of span	30 percent of deck weight accounts for haunch and interior diaphragms	30 percent of deck weight accounts for haunch and interior diaphragms
N_{harp}	10	12	11
$N_{straight}$	24	22	22
N_{temp}	4	4	4
e_{He}	8.00 in.	26.91 in.	30.61 in.
e_{Hcl}	9.00 in.	3.00 in.	3.00 in.
e_{Hbcl}	9.00 in.	3.00 in.	3.00 in.
e_S	3.50 in.	3.27 in.	3.27 in.
M_u	6903 kip-ft	6220 kip-ft	6019 kip-ft
M_s	5035 kip-ft	4085 kip-ft	3916 kip-ft

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 kip-ft = 1356 N-m.

W74G Girder Designs

Girder:

A_{gir} = 747 sq in. (481 935 mm^2)
I_{gir} = 547,553 in.^{4} (2.279 × 10^{11} mm^4)
w_{gir} = 830 plf (1235 kg/m)
h_{gir} = 73.5 in. (1867 mm)
y_t = 35.47 in. (901 mm)
y_b = 38.03 in. (966 mm)
b_{tf} = 43.00 in. (1092 mm)
b_w = 6.00 in. (152 mm)
f'_{ci} = 7.0 ksi (48 MPa)
f'_c = 8.0 ksi (55 MPa)

Slab:

t_s = 7.50 in. (190 mm)
t_{se} = 7.00 in. (178 mm)
f'_c = 4.0 ksi (28 MPa)

Prestressing:

A_{strand} = 0.217 sq in. (140 mm^2)
F_{pu} = 270.0 ksi (1862 MPa)
F_{pi} = 202.5 ksi (1396 MPa)
F_{py} = 243.0 ksi (1675 MPa)
E_{ps} = 28,500 ksi (196,500 MPa)
Low relaxation strand

Miscellaneous:

H = 80
PPR = 1.0
$t_{transfer}$ = 1 day
w_{curb} = 150 plf (223 kg/m)
$w_{overlay}$ = 25 psf (122 kg/m^2)
$w_{concrete}$ = 160 pcf (2563 kg/m^3)
Harping point = 0.4L

W74G	Simple Span Design	Continuous Span Design (Longer Span)	Continuous Span Design (Wider Girder Spacing)
Span length	127.5 ft	138.0 ft	127.5 ft
Girder spacing	8.0 ft	8.0 ft	9.1425 ft
Haunch	4.5 in.	—	—
Interior diaphragms	43.0 in. x 8.0 in. at 1/4 points of span	30 percent of deck weight accounts for haunch and interior diaphragms	30 percent of deck weight accounts for haunch and interior diaphragms
N_{harp}	15	17	16
$N_{straight}$	24	24	24
N_{temp}	6	6	6
e_{He}	30.53 in.	35.21 in.	38.10 in.
e_{Hcl}	3.60 in.	3.88 in.	3.75 in.
e_{Hbcl}	3.00 in.	3.00 in.	3.00 in.
e_S	3.38 in.	3.50 in.	3.50 in.
M_u	10,293 kip-ft	9456 kip-ft	8993 kip-ft
M_s	7669 kip-ft	6342 kip-ft	5971 kip-ft

Note: 1 ft = 0.3048 m; 1 in. = 25.4 mm; 1 kip-ft = 1356 N-m.

W83G Girder Designs

Girder:

A_{gir} = 972 sq in. (627,095 mm^2)
I_{gir} = 956,329 in.^{4} (3.981 × 10^{11} mm^4)
w_{gir} = 1080 plf (1607 kg/m)
h_{gir} = 82.68 in. (2100 mm)
y_t = 43.02 in. (1093 mm)
y_b = 39.66 in. (1007 mm)
b_{tf} = 49.02 in. (1245 mm)
b_w = 6.10 in. (155 mm)
f'_{ci} = 7.0 ksi (48 MPa)
f'_c = 8.0 ksi (55 MPa)

Slab:

t_s = 7.50 in. (190 mm)
t_{se} = 7.00 in. (178 mm)
f'_c = 4.0 ksi (28 MPa)

Prestressing:

A_{strand} = 0.217 sq in. (140 mm^2)
F_{pu} = 270.0 ksi (1862 MPa)
F_{pi} = 202.5 ksi (1396 MPa)
F_{py} = 243.0 ksi (1675 MPa)
E_{ps} = 28,500 ksi (196,500 MPa)
Low relaxation strand

Miscellaneous:

H = 80
PPR = 1.0
$t_{transfer}$ = 1 day
w_{curb} = 150 plf (223 kg/m)
$w_{overlay}$ = 25 psf (122 kg/m^2)
$w_{concrete}$ = 160 pcf (2563 kg/m^3)
Harping point = 0.4L

W83G	Simple Span Design	Continuous Span Design (Longer Span)	Continuous Span Design (Wider Girder Spacing)
Span length	158.5 ft	172.0 ft	158.5 ft
Girder spacing	8.5 ft	8.5 ft	9.714 ft
Haunch	4.5 in.	—	—
Interior diaphragms	61.0 in. x 10.0 in. at 1/4 points of span	30 percent of deck weight accounts for haunch and interior diaphragms	30 percent of deck weight accounts for haunch and interior diaphragms
N_{harp}	20	23	21
$N_{straight}$	40	40	40
N_{temp}	6	6	6
e_{He}	13.00 in.	29.25 in.	28.27 in.
e_{Hcl}	4.20 in.	4.36 in.	4.22 in.
e_{Hbcl}	3.00 in.	3.00 in.	3.00 in.
e_S	3.60 in.	3.54 in.	3.54 in.
M_u	16,775 kip-ft	15,764 kip-ft	14,790 kip-ft
M_s	12,813 kip-ft	10,847 kip-ft	10,076 kip-ft

Note: 1 ft = 0.3048 m; 1 in. = 25.4 mm; 1 kip-ft = 1356 N-m.

Restraint Moment Data

The software program RMCalc was used for calculation of restraint moments. All calculations are based on a five-span bridge with a deck thickness of 7.5 in. (190 mm). The ratio of strand harping length to span length is 0.4 for all girders.

The additional dead load accounts for a wearing surface load of 25 psf (122 kg/m^2) and a tributary traffic barrier load of 150 lbs per ft (223 kg/m). The haunch and interior diaphragm weight is accounted for by increasing the unit weight of the deck concrete by 30 percent. Therefore, the deck concrete unit weight is 201.5 pcf (3228 kg/m^3) [= 155 pcf (2483 kg/m^3) x 1.3]. Girder concrete unit weight was taken as 160 pcf (2563 kg/m^3).

Girder ages were used as follows:

Girder Age at Prestress Release = 1 day

Girder Age at Continuity = 30 days, 60 days, 90 days

Girder Age at Time Deck is in Place = 16 days, 46 days, 76 days

See "Ultimate Creep and Shrinkage Coefficients" and W58G Girder Designs – "Continuous Span Design (Longer Span)" for creep and shrinkage data.

Other data for calculation of restraint moments is shown to the right.

	W58G	W74G	W83G
B1	25.0 in.	43.0 in.	49.0 in.
B2	25.0 in.	25.0 in.	38.375 in.
B3	6.0 in.	6.0 in.	6.125 in.
B4	0	2.0 in.	3.0 in.
D1	58.0 in.	73.5 in.	82.68 in.
D2	5.0 in.	2.875 in.	3.0 in.
D3	2.0 in.	2.625 in.	3.0 in.
D4	0.0	2.0 in.	3.0 in.
D5	3.0 in.	3.0 in.	5.866 in.
D6	6.0 in.	6.0 in.	5.118 in.
Span length	115.0 ft	138.0 ft	172.0 ft
Pier diaphragm width	4.0 ft	5.0 ft	6.0 ft
Girder spacing	7.5 ft	8.0 ft	8.5 ft
Additional dead load	45 psf	43.75 psf	42.65 psf

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 psf = 4.882 kgf/m^2.

APPENDIX B — NOTATION

A_{gir} = cross-sectional area of girder
A_{strand} = cross-sectional area of prestressing strand
b_{tf} = width of top flange of girder
b_w = width of girder web
B1, B2, B3, B4 = width dimensions, as indicated in Fig. B1
D1, D2, D3, D4, D5, D6 = depth dimensions, as indicated in Fig. B1
e_{Hbcl} = center of gravity of lower harped strand bundle at centerline of girder, measured from the bottom of the girder
e_{Hcl} = center of gravity of harped strands at centerline of girder, measured from the bottom of the girder
e_{He} = center of gravity of harped strands at end of girder, measured from the top of the girder
e_S = center of gravity of straight strands, measured from the bottom of the girder
E_{ps} = elastic modulus of prestressing strand
f'_c = strength of concrete at 28 days
f'_{ci} = strength of girder concrete at release of prestressing
F_{pu} = ultimate strength of prestressing strand
F_{pi} = stress in prestressing strand at release of prestressing
F_{py} = yield strength of prestressing strand
h_{gir} = height of girder
H = percent ambient relative humidity
I_{gir} = moment of inertia of girder for bending
M_s = total service midspan moment
M_u = total factored midspan moment
N_{harp} = number of harped strands
$N_{straight}$ = number of straight strands
N_{temp} = number of temporary top strands
PPR = partial prestressing ratio
t_s = total thickness of deck slab
t_{se} = thickness of deck slab used for design
$t_{transfer}$ = time from when girder is cast to when prestressing strands are released

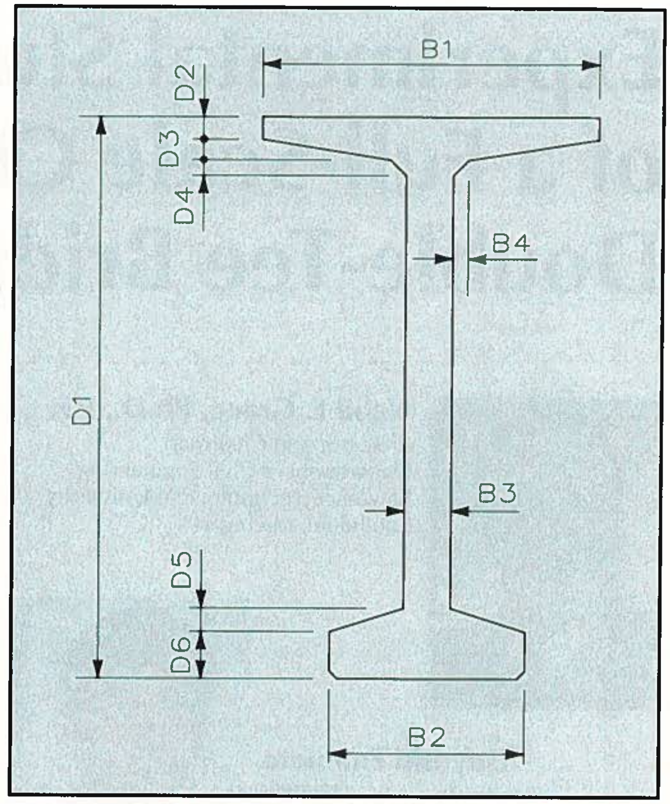

Fig. B1. Girder dimensions for restraint moment calculations.

$w_{concrete}$ = unit weight (mass) of concrete
w_{curb} = tributary weight (mass) of curb
w_{gir} = weight (mass) of girder
$w_{overlay}$ = weight (mass) of superimposed overlay
y_t = distance from top of girder to center of gravity
y_b = distance from bottom of girder to center of gravity
γ_{creep} = correction factor for computing ultimate creep coefficient
$\gamma_{shrinkage}$ = correction factor for computing ultimate shrinkage coefficient