How long is long? An analytical study of precast, prestressed concrete girder spans using 0.7 in. strand

Kent A. Harries, Bahram M. Shahrooz, Payne D. Ball, TianQiao Liu, Venkata S.S.P. Sathiraju, Abdullah Alabdulkarim, Richard A. Miller, and Reid W. Castrodale

- An extensive analytical study is presented to assess the maximum girder span lengths that can be achieved when using 0.6 and 0.7 in. (15.2 and 17.8 mm) strands.
- Girder span increases of up to 22% were achieved using 0.7 in. strand in place of 0.6 in. strand.
- The larger pretension forces affected end-region detailing and increased congestion, although all resulting requirements were constructable.

In the United States, seven-wire prestressing strands conforming to ASTM A416[1] and AASHTO M 203[2] are used to pretension concrete bridge components. Typically, Grade 270 (1860 MPa) low-relaxation strand is used for bonded pretensioning (referred to in this context as *prestressing strand* or *strand*). For many years, the standard in the bridge industry was 0.5 in. (13 mm) diameter strand. Research conducted in the 1990s supported the use of 0.6 in. (15.2 mm) diameter strand. This size of strand is now commonly used as a means of increasing available pretensioning force, which makes it possible to extend spans, increase girder spacing, and decrease structural depth. Currently, seven-wire, 0.7 in. (17.8 mm) diameter Grade 270 (1860 MPa) low-relaxation strands are primarily used as cable or strand roof anchors in the mining and tunneling industries; however, these strands, which conform to ASTM A416, may provide benefits similar to those associated with 0.6 in. strand. **Table 1** compares the properties of 0.5, 0.6, and 0.7 in. strands.

The use of 0.7 in. (17.8 mm) strands in pretensioned bridge components is not explicitly permitted in any known international design standard.[3] The American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[4] includes 0.7 in. strand by reference to AASHTO M 203[2] but is otherwise silent on the use of 0.7 in. strand. Internationally, ASTM A416[1] seems to be the only available specification for seven-wire, Grade 270 (1860 MPa) 0.7 in. strand. (The European standard EN 10138[5] identifies seven-wire, Grade 250 [1720 MPa] 0.7 in. strand products, but these are not used in

Designation, in.	Nominal diameter, mm	Nominal area, mm^2	Nominal weight, kg/m	Minimum breaking strength, kN	Minimum load at 1% extension, kN	Minimum elongation in 605 mm, %
0.500	12.7	99	0.77	184	165	3.5
0.600	15.2	140	1.10	261	234	
0.700	17.8	190	1.49	353	318	

Table 1. Physical properties of Grade 270 (1860 MPa) low-relaxation strand

Source: Data from ASTM International (2015).

Note: 1 mm = 0.0394 in.; 1 in. = 25.4 mm; 1 kN = 0.225 kip; 1 kg/m = 0.672 lb/ft.

bridge construction.) Nonetheless, Grade 270 0.7 in. strands have been evaluated as pretensioning reinforcement in several experimental projects.[6–8]

This study was conducted as part of National Cooperative Highway Research Program (NCHRP) project 12-109. Given the limitations on the length of this paper, the reader is referred to the NCHRP report[3] and appendixes[9] for additional study details.

Motivation for using 0.7 in. strands

Record-breaking span lengths have recently been achieved by using 0.6 in. (15.2 mm) strands. Examples include 62.5 m long (205 ft) girders in the Alaskan Way viaduct in Seattle, Wash.;[10] 63.7 m long (209 ft) girders in the U.S. 17-92 interchange at State Road 436 in Casselberry, Fla.;[11] 64.0 m long (210 ft) girders in the Deerfoot Trail extension near Calgary, AB, Canada;[12] and 68.0 m long (223 ft) modified wide-flange girders for a high-occupancy vehicle extension in Tacoma, Wash.[13] Given these accomplishments, a logical question is whether the precast concrete industry needs 0.7 in. diameter strands.

Table 1 shows that the area of a 0.7 in. (17.8 mm) strand is 92% greater than that of a 0.5 in. (13 mm) strand and 36% greater than a 0.6 in. (15.2 mm) strand. The larger area of 0.7 in. strands, in conjunction with higher-strength concrete, has the potential to offer the following benefits:[3]

- The number of strands required in a girder for the same girder span could be reduced. Fewer strands would alleviate congestion in heavily reinforced pretensioned elements and may be economically advantageous. However, simply reducing the number of strands in a girder by replacing 0.6 in. strand with fewer 0.7 in. strands that provide the same reinforcement area has little, if any, structural advantage, such as increasing span length or allowing for fewer girders (by increasing girder spacing). Reducing the required number of strands could be potentially beneficial in situations where filling all (or most) strand locations in a section with 0.6 in. strand does not provide sufficient pretensioning force. Replacing 0.6 in. strand with 0.7 in. strand on a one-to-one basis using the same 51 mm (2.0 in.) grid spacing is one way to achieve a greater pretension force.
- The total number of girders in a bridge could be reduced by using individual girders that have greater pretension force. Fewer girders may shorten the construction time, cut construction costs, and reduce overall energy consumption for fabrication and girder transportation.
- Longer spans could be achieved by using girders with greater pretension force. Longer spans may reduce the number of piers required for a new bridge or permit the elimination of the central pier in typical two-span bridges. In bridge replacement projects, particularly in congested urban areas, eliminating the central piers on large thoroughfares or interstate crossings may permit more efficient expansion of the roadways beneath the bridge, eliminate the hazards associated with piers located close to the roadways, and minimize the impact of the span on environmentally sensitive habitats. Nonetheless, there are practical upper limits on girder length based on size and weight limitations associated with shipping and handling.[14]
- Shallower girders that have greater pretension force could be used for the same span length. This benefit becomes particularly important in replacement projects that must increase existing clearances or expand the hydraulic opening beneath the bridge.

Objectives of reported study

The objectives of the analytical study presented in this paper were to assess maximum achievable pretensioned girder span lengths when using 0.6 and 0.7 in. (15.2 and 17.8 mm) strands and examine the influence of girder shape and size on the potential benefits of using 0.7 in. strands. The study also examined the impacts of using 0.7 in. strands on girder end region detailing requirements, prestress transfer, and the handling and erection stability of long-span girders.

This study did not specifically address many additional factors that should be considered regarding the use of 0.7 in. (17.8 mm) strands. Some of the issues that require consideration involve the handling of the heavier and stiffer strand and larger strand forces and the potential need to retool existing stressing beds and hardware.

Parametric study

The investigators generated 584 pretensioned simple-span girder design cases to compare girders using 0.6 and 0.7 in. (15.2 and 17.8 mm) strands. The objective of each design case was to maximize the girder span while respecting all requirements of the AASHTO LRFD specifications.[4] Issues of handling stability were not considered in the parametric study, but were subsequently analyzed for girders having the longest resulting span lengths, as described later in this paper.

Each design case was replicated with 0.6 and 0.7 in. (15.2 and 17.8 mm) Grade 270 (1860 MPa) prestressing strands. Ten girder types (cross sections) of varying depth were considered (**Table 2**).

Three normalweight concrete compressive strengths f'_c (69, 103, and 124 MPa [10, 15, 18 ksi]) and one lightweight concrete compressive strength f'_c(69 MPa) were considered. The unit weights of concrete w_c were 2400, 2480, 2530, and 2000 kg/m^3 (150, 155, 158, 125 lb/ft^3), respectively. The concrete strength at strand release f'_{ci} was assumed to be $0.6f'_c$ for cases where f'_c exceeded 69 MPa, and $0.8f'_c$ for cases where f'_c was equal to or less than 69 MPa.[6] A normalweight concrete composite slab with a compressive strength f'_c of 31 MPa (4.5 ksi) and unit weight w_c of 2320 kg/m^3 (145 lb/ft^3) was included in all designs. A nominal 80 kg/m^3 (5 lb/ft^3) allowance for reinforcing steel was added to all concrete unit weights.

When the single-web girder analyses were conducted, it was assumed that the girders were interior girders having the following combinations of spacing S and slab thickness t_f:

- S = 1830 mm (72.0 in.) and t_f = 203 mm (7.99 in.)
- S = 2440 mm (96.0 in.) and t_f = 203 mm (7.99 in.)
- S = 2540 mm (100 in.) and t_f = 229 mm (9.02 in.)
- S = 3660 mm (144 in.) and t_f = 229 mm (9.02 in.)

All slabs were provided with a 51 mm deep (2.0 in.) haunch.

Double-web cases considered the following:

- S = 3660 mm (144 in.) and t_f = 203 mm (8.00 in.)
- S = 4270 mm (168 in.) and t_f = 203 mm (8.00 in.)
- S = 4880 mm (192 in.) and t_f = 229 mm (9.02 in.)

Girder cross sections and key properties for all shapes are provided in NCHRP Web-Only Document 315 appendix A.[9]

The designs assumed a simple, nonskewed span, and the designs were performed using a spreadsheet developed by the authors, which was benchmarked and validated against LEAP

Table 2. Girder types and designations used in parametric study

Nominal depth, mm	Single web					Double web				
	PCI bulb-tee girder	AASHTO I-girder	FIB girder	Ohio WF girder	NU girder	Washington WF I-girder	AASHTO box girder	NEXT deck beam	Texas U girder	Washington U girder
915				WF36	NU900					
1020								40D	U40	
1100					NU1100					
1220							BIV48			
1370	BT54			WF54					U54	U54G5
1520										UF60G5
1600					NU1600					
1625	BT63									
1830	BT72	VI		WF72						UF72G5
1880						WF74G				
2000					NU2000					
2440			FIB96							
2540						WF100G				

Note: FIB = Florida I-beam; NEXT = northeast extreme tee; NU = University of Nebraska I-girder; WF = wide flange. 1 mm = 0.0394 in.

CONSPAN software from Bentley Systems. The design methodology used to determine the greatest achievable span is described in the next section of this paper. Live load distribution to the interior girders was determined in accordance with the approximate analysis method of article 4.6.2.2 of the AASHTO LRFD specifications.[4] Additional assumed dead loads included a 1.2 kPa (25 lb/ft^2) allowance for wearing surface *DW* and a 358 kg/m (241 lb/ft) allowance added to each girder for barrier walls and appurtenances *DC*.

Design procedure for parametric study

The following steps summarize the design procedure for determining the longest achievable span for each design case. These steps were programmed into the PCI Girder Stability Analysis version 1.0 Excel spreadsheet.

Step 1

The girder section is filled with as many straight strands as geometrically possible, and the span is increased until either the Service I or Service III limits of the AASHTO LRFD specifications[4] are reached at midspan or the Strength I limit is achieved; from this step, the maximum achievable span is determined. To achieve these limits, four partially tensioned top strands providing a total of 267 kN (60.0 kip) compressive force are used for all cases. These top strands are not included in Strength I limit state calculations as is customary for prestressing strand on the compression side of a girder.

Step 2

The tensile and compressive stress limits at prestress transfer and the Service I and Service III limits near the end of the girder are checked. If these stress limits are not exceeded, the design is complete based on step 1. Otherwise, the design progresses to step 3.

If the stress limits at transfer are exceeded, the value of concrete strength at prestress transfer $f'_{ci} = \eta f'_c$ is increased until the limits are satisfied. This side check is akin to permitting the girder to cure further before prestress transfer to mitigate excessive stresses. The values used for design are f'_{ci} equal to $0.8f'_c$ for $f'_c \leq 69$ MPa (10 ksi) and f'_{ci} equal to $0.6f'_c$ for $f'_c > 69$ MPa. Therefore, a value of η greater than 1.0 indicates that the design is controlled by concrete strength at release, and f'_c must be increased or the span shortened. A value of η between 0.8 (or 0.6) and 1.0 indicates that the stress limits could be met by delaying release to increase concrete strength at transfer. Regardless of the outcome of this side check, all designs in this study progressed using the values of f'_{ci} equal to $0.8f'_c$ or $0.6f'_c$.

Step 3

Using the design values for f'_{ci}, strand debonding is attempted to satisfy the step 2 stress check(s) while limiting the total debonding ratio *dr* to 25% or less (≤ 40% in any single-strand layer). If the step 2 stress checks are satisfied by this debonding, the design is complete.

Step 4

If a successful design is not possible using debonding (step 3), all strands are assumed to be bonded, and harping is attempted to satisfy the step 2 stress limits. Regardless of girder length, harp points are assumed to be 4.6 m (15 ft) to either side of the girder centerline. If more than eight strands must be harped, second harp points are selected 5.8 m (19 ft) to either side of the girder centerline. If the stress checks are satisfied by harping, the design is complete.

Step 5

If harping alone (step 4) is insufficient, a combination of harping and debonding is used to bring the step 2 stresses within limits. The number of harped strands is limited to as few as possible. If the stress checks are satisfied by a combination of harping and debonding, the design is complete. Note that Texas U girders are not permitted to have harped strands. Therefore, debonding strands is the only available method for keeping release stresses below the AASHTO LRFD specification limits for this girder type.

Step 6

If the methods of mitigating stresses considered in steps 3, 4, or 5 remain insufficient to satisfy the step 2 stress limits, the span length is reduced (in increments of 305 mm [1 ft]), and the process is repeated until a design satisfying all stress limits is achieved.

Step 7

Once an acceptable span is obtained, the required transverse reinforcement is determined.

Step 8

A final design constraint is based on satisfying requirements of AASHTO LRFD specifications articles 5.9.4.4.1 and 5.9.4.4.2 dealing with splitting reinforcement and confinement near the ends of girders, as well as requirements for longitudinal reinforcement in AASHTO LRFD specifications article 5.7.3.5. The designs limit splitting reinforcement near the ends of the girders to no. 5 (16M) reinforcement, with spacing equal to or greater than 51 mm (2.0 in.).

The design steps were used to first design a girder using 0.6 in. (15.2 mm) strands for the maximum possible span at the minimum considered spacing *S* of 1830 mm (72.0 in.) for single-web girders, or *S* of 3660 mm (144 in.) for double-web girders. New girders were designed using the same concrete strength, concrete density, and strand diameter while increasing the girder spacing in 610 mm (24 in.) increments. The process was then repeated with 0.7 in. (17.8 mm) strands.

Results of parametric study

The maximum achievable spans for pretensioned girders using 0.6 and 0.7 in. (15.2 and 17.8 mm) strands ($L_{0.6}$ and $L_{0.7}$, respectively) were computed. The resulting design details of all 584 cases—including the number of straight and harped strands, the number and length of debonded strands, and required web reinforcement—are provided in NCHRP Web-Only Document 315 appendix B.[9] **Figure 1** presents a representative resulting span length chart for PCI BT72 (72 in. [1800 mm] bulb tee). NCHRP Web-Only Document 315 appendix D has similar charts for all 10 girder shapes considered. Figure 1 indicates the percentage increase in the achievable span length resulting from the use of 0.7 in. strands, and **Table 3** summarizes this for all girder types. For the PCI BT72 case (Fig. 1), a greater potential increase in span length is associated with higher concrete compression strength and increased girder spacing.

Potential increase in span length

Figure 2 compares the maximum span lengths achieved using 0.6 and 0.7 in. (15.2 and 17.8 mm) strands. When 0.7 in. strands were used, the achievable span lengths increased within a band of 1.0 to 1.22 times their corresponding design cases having 0.6 in. strands. Therefore, by using 0.7 in. strand, it was possible to increase the span length by a maximum of 22% in comparison to designs using 0.6 in. strand. The largest value of 1.22 is less than the ratio of the area of a 0.7 in. strand to that of a 0.6 in. strand, 1.35 (Table 1), which indicates that a one-to-one replacement of strands is not possible given the many other design constraints (such as stress limits at prestress release) that must be considered. Thus, the full potential benefit of using 0.7 in. strands cannot be realized for the sections and designs considered.

Influence of girder geometry

The potential for increasing the girder span using 0.7 in. (17.8 mm) strands was affected by girder geometry (Table 3 and Fig. 2). In general, girders whose design had been optimized based on 0.6 in. (15.2 mm) strands (for example, University of Nebraska [NU] and wide-flange [WF] girders) offered little potential for span length increases using 0.7 in. strands. More general sections (PCI bulb-tee girders) and deep sections established by lengthening the web between standard top and bottom bulbs (Washington 100 in. [2540 mm] wide-flange I-girder [WF100G]) showed greater potential for increasing span lengths using 0.7 in. strand.

In almost all cases, when 0.7 in. (17.8 mm) strand was used, more harped strands and/or greater debonding were required because the total pretension force was greater. Harped strands are not permitted in Texas U girders; as a result, the potential increase in span was limited (Table 3).

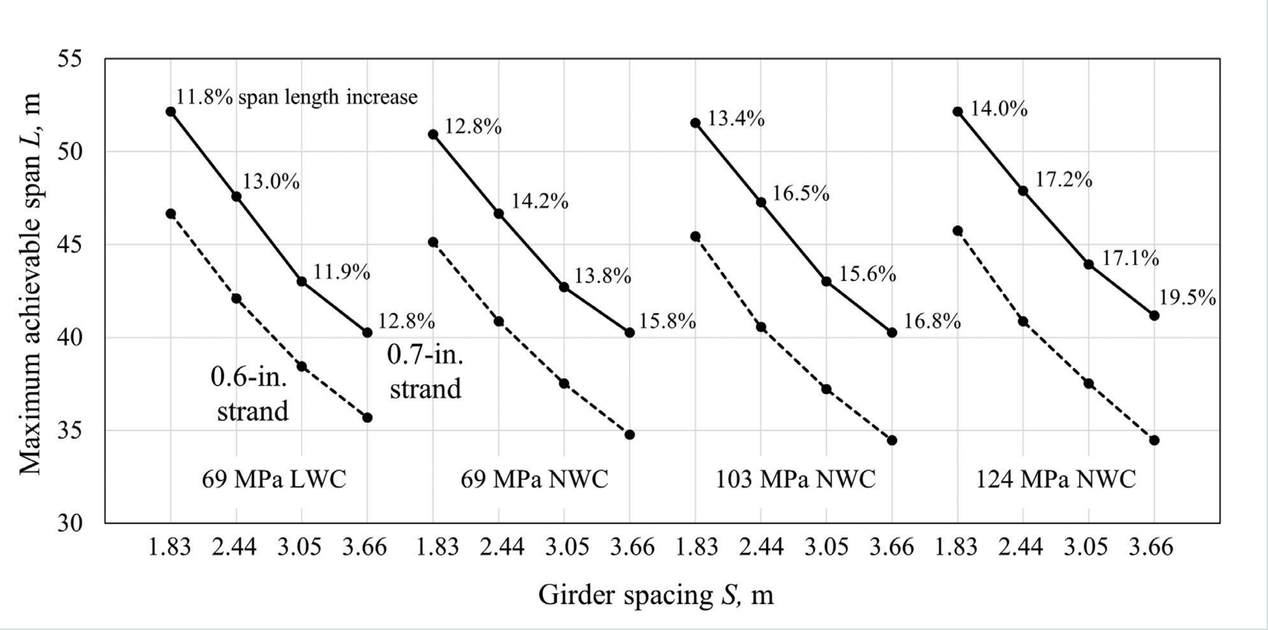


Figure 1. Maximum span length chart for PCI BT72 with different concrete strengths. The percentages shown indicate increases in achievable span length when 0.6 in. diameter strands (lower curves with dashed lines) are replaced with 0.7 in. diameter strands (upper curves with solid lines). Note: BT72 = 72 in. bulb tee; LWC = lightweight concrete; NWC = normalweight concrete. 1 in. = 25.4 mm; 1 m = 3.28 ft; 1 MPa = 0.145 ksi.

Table 3. Ratios of achievable span lengths using 0.7 in. diameter strands to achievable span lengths using 0.6 in. diameter strands $L_{0.7}/L_{0.6}$

Girder	Average	Minimum	Maximum
PCI 54 in. bulb tee (BT54)	1.10	1.03	1.15
PCI 63 in. bulb tee (BT63)	1.13	1.09	1.17
PCI 72 in. bulb tee (BT72)	1.15	1.12	1.19
AASHTO I-girder Type VI	1.06	1.01	1.13
Florida 96 in. I-beam girder (FIB96)	1.10	1.06	1.13
Washington 74 in. wide-flange I-girder (WF74G)	1.14	1.06	1.21
Washington 100 in. wide-flange I-girder (WF100G)	1.18	1.13	1.22
University of Nebraska 900 mm I-girder (NU900)	1.02	1.00	1.05
University of Nebraska 1100 mm I-girder (NU1100)	1.03	1.00	1.05
University of Nebraska 1600 mm I-girder (NU1600)	1.06	1.01	1.12
University of Nebraska 2000 mm I-girder (NU2000)	1.09	1.05	1.12
Ohio 36 in. wide-flange girder (WF36)	1.02	1.01	1.02
Ohio 54 in. wide-flange girder (WF54)	1.07	1.06	1.08
Ohio 72 in. wide-flange girder (WF72)	1.12	1.11	1.13
AASHTO 48 in. box IV girder (BIV48)	1.08	1.06	1.09
Northeast extreme tee 40 in. deck beam (NEXT40D)	1.15	1.15	1.16
Texas 40 in. U girder (U40)	1.02	1.01	1.04
Texas 54 in. U girder (U54)	1.05	1.02	1.11
Washington 54 in. U girder (U54G5)	1.11	1.02	1.18
Washington 60 in. U girder (UF60G5)	1.18	1.18	1.19
Washington 72 in. U girder (UF72G5)	1.18	1.17	1.19

Note: 1 in. = 25.4mm; 1 mm = 0.0394 in.

One measure of the efficiency of a prestressed concrete section is the lever arm of the internal couple.[15] This value is represented as $e + k_t$ (**Fig. 3**), where e is the distance between the centroid of the cross section and the centroid of prestressing steel and k_t is the distance between the centroid of the cross section and the top kern point. The top kern point is defined as the uppermost location in the cross section at which the compression resultant may be placed such that the condition of zero tension is maintained at the bottom face of the girder. The greater the distance $e + k_t$ is, the more efficient the section is, and so less prestressing force is required to carry a given load over a given span. The value $e + k_t$ was calculated for all sections considered herein and normalized with respect to girder depth h. Figure 3 plots the relationship between the achievable span increase and $(e + k_t)/h$ for all the sections considered. Less-efficient girders—that is, those with a smaller $(e + k_t)/h$ (such as Washington U girders [UF] and PCI BT72 girders)—tend to benefit more from using 0.7 in. (17.8 mm) strands.

In addition to the girder geometry, the achievable increase in span length using 0.7 in. (17.8 mm) strands generally increased as a function of girder depth (Fig. 3). Even among relatively optimized shapes, deeper girders had a greater potential relative increase in span length when using 0.7 in. strands (for example, compare NU girders in Table 3 and Fig. 3).

Although there were variations in the resulting designs,[3,9] girders designed for a greater spacing S generally had a greater potential for span increase (Fig. 1). This finding is associated with the proportionally lower live load distribution factor inherent as S increases.[4] Greater potential increases in girder span were also observed as concrete strength increased. This finding reflects the fact that service-limit states—typically at prestress transfer—often control the design; higher concrete strength increases the design limits on concrete stresses.

Comparison of observations with previous related study

Salazar et al.[16] published a parametric study on the use of 0.7 in. (17.8 mm) strands that was similar to the study reported herein. Using a methodology like that adopted here, Salazar et al. considered AASHTO Type IV, Type V, and Type VI girders, Texas bulb-tee girders, Texas spread-box girders, and Texas U girders. They concluded that I-girder and bulb-tee girder spans could be increased up to 3 m (10 ft) by using 0.7 in. strand instead of 0.6 in. (15.2 mm) strand. This increase generally required concrete strength at prestress transfer f'_{ci} greater than 69 MPa (10 ksi) and the use of harping or other methods to control end-region stresses. Salazar et al. concluded that use of 0.7 in. strand in U girders and box girders did not result in greater span lengths than could be achieved with 0.6 or 0.5 in. strands. They concluded that for I-girders and bulb-tee girders of a given span length, the required depth of some of the girder shapes could be decreased if 0.7 in. strands were used in conjunction with higher concrete strength at prestress transfer. Although many I-girders and bulb-tee girders reported in that study achieved longer spans when 0.7 in. strands were used, not all did. Contrary to our findings, Salazar et al. reported that shallower cross sections benefitted more from the use of larger strand diameters. Also, somewhat contrary to the findings of this study, Salazar et al. concluded that there was little advantage to using 0.7 in. strand to increase girder spacing and that any advantage appeared at a girder spacing so large as to be impractical. Nonetheless, in general, the conclusions of Salazar et al. are consistent with the findings of our study, with one notable exception; Salazar et al. found no advantage to using 0.7 in. strand in box girders, whereas our study did indicate a potential advantage. Differences in box-girder configuration may explain this discrepancy. Salazar et al. only considered Texas box shapes used in a spread configuration, whereas this study considered AASHTO box shapes in an adjacent configuration.

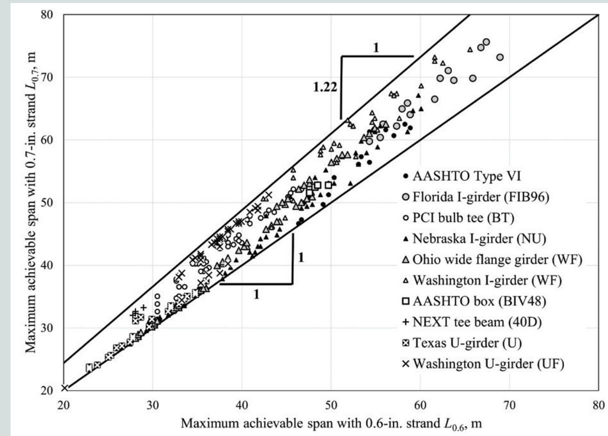
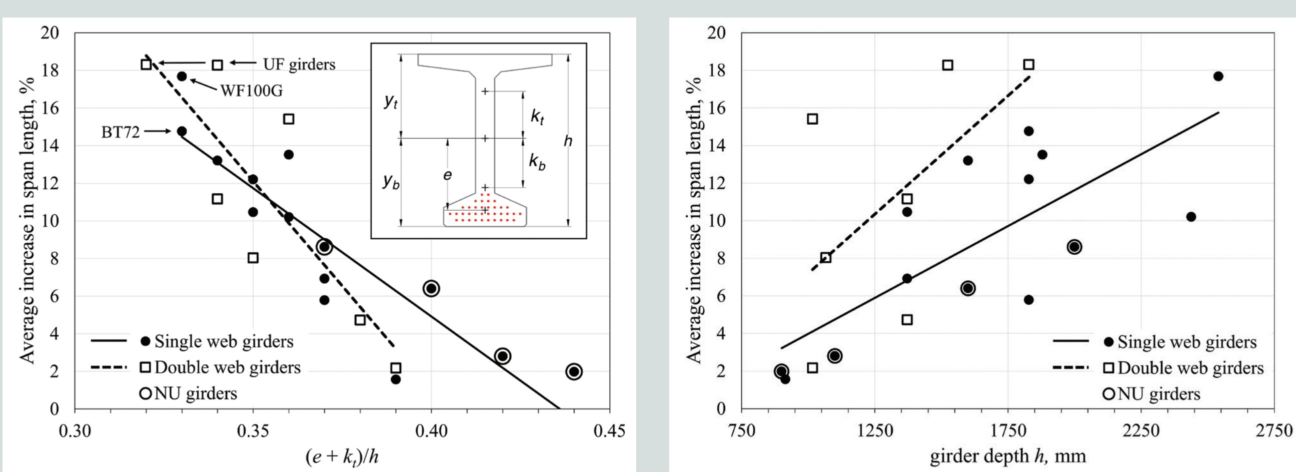


Figure 2. Comparison of maximum achievable span lengths using 0.6 in. and 0.7 in. diameter strands. Note: 1 in. = 25.4 mm; 1 m = 3.28 ft.



Effect of normalized girder efficiency $(e + k_t)/h$

Effect of girder depth h

Figure 3. Achievable span length increases as function of girder geometry. Note: BT72 = 72 in. (1800 mm) bulb tee; e = distance between the centroid of the cross section and the centroid of prestressing steel; k_t = distance between the centroid of prestressing steel and the top kern point; NU = University of Nebraska I-girder; UF = Washington U girder; WF100G = Washington wide-flange 100 in. (2540 mm) I-girder. 1 mm = 0.0394 in.

Design validation by finite element modeling of full-length girders

To validate the approach used in the parametric design study reported herein, investigators used the 2016 release of Cervenka Consulting's ATENA finite element software to develop full-girder models of selected cases. The analyses were based on the extensive modeling and validation studies presented in Shahrooz et al.[6] Material properties used for the models were consistent with those used for design (as discussed previously). Initial prestressing force f_{pi} was taken as 1396 MPa (202.5 ksi), and transfer length was consistent with the AASHTO LRFD specifications.[4] Transfer length L_t is equal to $60d_b$ (where d_b is the strand diameter) or 915 mm (36.00 in.) for 0.6 in. (15.2 mm) strand and 1067 mm (42.01 in.) for 0.7 in. (17.8 mm) strand. Prestress losses upon tendon release were determined within the finite element model based on the bond slip model; these losses were approximately 10% of the initial prestressing force but varied somewhat based on the girder cross section.[6] Long-term losses were calculated to result in an effective prestressing force of $0.56f_{pu}$ (where f_{pu} is the tensile strength of the strand)[6] and were used at all subsequent steps (**Table 4**). Only the critical flexural load case was considered in steps 3, 4, and 5. The overstrength factor Ω was found by increasing the axle loads LL_{truck} (only) until failure of the girder.

Two I-girder shapes were considered, and comparative designs with both 0.6 and 0.7 in. (15.2 and 17.8 mm) strands were modeled. PCI BT72 girders benefitted the most (in terms of potential span length increase) from replacing 0.6 in. strands with 0.7 in. strands. In contrast, 2000 mm (79 in.) University of Nebraska (NU2000) girders did not show great potential increases in span length (Table 3).

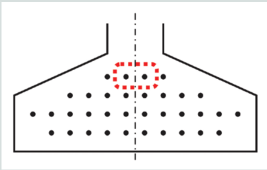
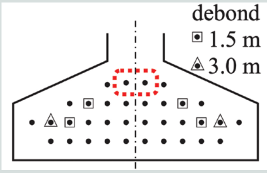
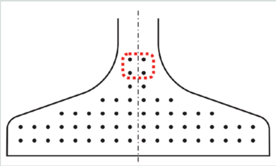
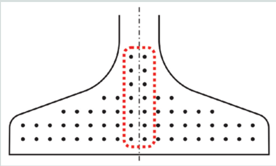
For each of the four cases (each girder shape using 0.6 or 0.7 in. [15.2 or 17.8 mm] strands), a girder spacing S equal to 2440 mm (96.1 in.) was used; deck tributary area and live load distribution factors were calculated on this basis. It was assumed that each girder was fabricated 450 mm (18 in.) longer than its span length L and was supported on 450 mm long, full-width neoprene bearings. The distribution factor for flexure of interior girders $g_{M,int}$ was determined from AASHTO LRFD specifications Table 4.6.2.2.2b-1 for cross section type (k). **Table 5** summarizes the details of the PCI BT72 and NU2000 models.

Table 6 summarizes the analysis results, and **Fig. 4** shows selected stress and crack distributions. The finite element results accurately mirrored the design requirements, and each girder mostly met the concrete stress requirements of the AASHTO LRFD specifications. At prestress transfer, the concrete tension stresses for the PCI BT72 girders fell between the AASHTO LRFD specification limits of $0.25\sqrt{f'_{ci}}$ ($0.095\sqrt{f'_{ci}}$ in ksi units) and $0.63\sqrt{f'_{ci}}$ ($0.24\sqrt{f'_{ci}}$ in ksi units), indicating a need for nonprestressed reinforcement in the region of tensile stress. The tensile stresses in the NU2000 girders at prestress transfer fell below $0.25\sqrt{f'_{ci}}$, so no additional reinforcement would be required. In all cases except PCI BT72-6 (where 6 indicates 0.6 in. [15.2 mm] diameter strand), the maximum compression stresses at prestress transfer exceeded the AASHTO LRFD specifications limit of $0.60f'_{ci}$. The high compressive stresses were localized near the girder ends. This discrepancy between the finite element model and section-based design

Table 4. Full-girder finite element modeling steps

Step	Description	Applied loads	Girder concrete, MPa	Slab concrete, MPa	Prestress, MPa
1	Release tendons	Girder self-weight only	$f_{ci} = 0.6f'_c = 62$ $E_{ci} = 41,000$	n/a	$\approx 0.9(0.75f_{pu}) = 182$
2	Place deck slab	Girder and slab self-weight	$f'_c = 103$ $E_c = 49,000$	n/a	$0.56f_{pu} = 151$
3	Service I	$DC + DW + (LL + IM)$	$f'_c = 103$ $E_c = 49,000$	$f'_c = 31$ $E_c = 30,000$	$0.56f_{pu} = 151$
	Service III	$DC + DW + 0.8(LL + IM)$			
4	Strength	$1.25DC + 1.50DW + 1.75(LL + IM)$	$f'_c = 103$ $E_c = 49,000$	$f'_c = 31$ $E_c = 30,000$	$0.56f_{pu} = 151$
5	Failure	$1.25DC + 1.50DW + 1.75LL_{lane} + \Omega(LL_{truck} + IM)$	$f'_c = 103$ $E_c = 49,000$	$f'_c = 31$ $E_c = 30,000$	$0.56f_{pu} = 151$

Note: DC = weight of components (barrier walls and appurtenances); DW = weight of wearing surface; E_c = modulus of elasticity of concrete corresponding to f'_c ; E_{ci} = modulus of elasticity of concrete corresponding to f'_{ci} ; f'_c = concrete compressive strength; f'_{ci} = concrete strength at strand release; f_{pu} = tensile strength of prestressing strand; IM = impact factor; LL = HL93 live load; LL_{lane} = lane load component of HL93 live load; LL_{truck} = truck axle load component of HL93 live load; n/a = not applicable; Ω = overstrength factor. 1 MPa = 0.145 ksi.

<table>
<tr><th colspan="9">Table 5. Details of PCI BT72 and NU2000 full-girder finite element models</th></tr>
<tr><th>Model</th><th colspan="2">PCI BT72-6</th><th colspan="2">PCI BT72-7</th><th colspan="2">NU2000-6</th><th colspan="2">NU2000-7</th></tr>
<tr><td>Strand, in.</td><td colspan="2">0.6</td><td colspan="2">0.7</td><td colspan="2">0.6</td><td colspan="2">0.7</td></tr>
<tr><td>Girder length, m</td><td colspan="2">40.4</td><td colspan="2">47.7</td><td colspan="2">55.4</td><td colspan="2">61.8</td></tr>
<tr><td>Girder span L, m</td><td colspan="2">40.0</td><td colspan="2">47.3</td><td colspan="2">54.9</td><td colspan="2">61.3</td></tr>
<tr><td>$g_{M.int}$</td><td colspan="2">0.66</td><td colspan="2">0.63</td><td colspan="2">0.63</td><td colspan="2">0.61</td></tr>
<tr><td>Strands at midspan (harped strands shown in box)</td><td colspan="2">
32 straight
2 harped</td><td colspan="2">
32 straight
2 harped
6 debonded</td><td colspan="2">
56 straight
4 harped</td><td colspan="2">
46 straight
14 harped</td></tr>
<tr><td rowspan="2">Location of harped strand pairs above soffit y, mm</td><td>x = 0</td><td>x = 15.6 m</td><td>x = 0</td><td>x = 19.3 m</td><td>x = 0</td><td>x = 23.1 m</td><td>x = 0</td><td>x = 26.3 m</td></tr>
<tr><td>1778</td><td>203</td><td>1778</td><td>203</td><td>1948
1897</td><td>356
305</td><td>1948
1897
1847
1796
1745
1694
1643</td><td>356
305
254
203
152
102
51</td></tr>
<tr><td>Top straight strands</td><td colspan="4">Four strands stressed to 67 kN at y = 1778 mm</td><td colspan="4">Four strands stressed to 67 kN at y = 1948 mm</td></tr>
<tr><td>$0.04A_{ps}f_{pi}$, kN</td><td colspan="2">267</td><td colspan="2">298</td><td colspan="2">467</td><td colspan="2">636</td></tr>
<tr><td>Web reinforcement at girder end</td><td colspan="2">Five pairs no. 5 at 102 mm</td><td colspan="2">Six pairs no. 5 at 76 mm</td><td colspan="2">Ten pairs no. 5 at 51 mm</td><td colspan="2">Thirteen pairs no. 5 at 51 mm</td></tr>
<tr><td>Web reinforcement over span</td><td colspan="2">Pairs no. 4 at 356 mm</td><td colspan="2">Pairs no. 4 at 457 mm</td><td colspan="2">Pairs no. 4 at 356 mm</td><td colspan="2">Pairs no. 4 at 356 mm</td></tr>
<tr><td>Bulb tie force, kN</td><td colspan="2">512</td><td colspan="2">391</td><td colspan="2">867</td><td colspan="2">1561</td></tr>
<tr><td>Bulb reinforcement at girder end</td><td colspan="2">Nine no. 3 hoops at 114 mm</td><td colspan="2">No. 3 hoops at 152 mm</td><td colspan="2">Nine no. 4 hoops at 121 mm</td><td colspan="2">Fifteen no. 4 hoops at 64 mm</td></tr>
<tr><td>Bulb reinforcement over span</td><td colspan="2">No. 3 hoops at 152 mm</td><td colspan="2">No. 3 hoops at 152 mm</td><td colspan="2">No. 3 hoops at 152 mm</td><td colspan="2">No. 3 hoops at 152 mm</td></tr>
</table>

Note: A_{ps} = area of prestressing strand; BT72-6 = 72 in. bulb tee with 0.6 in. strand; BT72-7 = 72 in. bulb tee with 0.7 in. strand; f_{pi} = initial prestressing force; $g_{M.int}$ = distribution factor for flexure of interior girders; NU200-6 = 2000 mm University of Nebraska I-girder with 0.6 in. strand; NU2000-7 = 2000 mm University of Nebraska (NU) I-girder with 0.7 in. strand; x = distance measured from end of girder; y = vertical location of prestressing strand measured from bottom of girder. No. 3 = 10M; no. 4 = 13M; no. 5 = 16M. 1 mm = 0.039 in.; 1 in. = 25.4 mm; 1 m = 3.28 ft; 1 kN = 0.225 kip.

Table 6. Summary of full-girder finite element analysis results					
Limit state	Parameter	PCI BT72-6	PCI BT72-7	NU2000-6	NU2000-7
Release of tendons	$f_{t,max}$, MPa	2.00	2.96	1.03	0.34
	$f_{c,max}$, MPa	35.4	54.5	43.1	46.9
	$w_{cr,max}$, mm	≤0.30	≤0.50	≤0.30	≤0.40
Service I	$f_{t,max}$, MPa	1.51	2.00	1.72	0.69
	$f_{c,max}$, MPa	32.4	54.1	40.6	42.7
	$w_{cr,max}$, mm	≤0.30	≤0.50	≤0.20	≤0.40
Service III	$f_{t,max}$, MPa	0.69	0.62	0.48	0.62
	$f_{c,max}$, MPa	32.6	54.1	40.7	43.0
	$w_{cr,max}$, mm	≤0.30	≤0.50	≤0.20	≤0.40
Strength I	$f_{t,max}$, MPa	4.69	5.79	6.00	6.41
	$f_{c,max}$, MPa	30.7	53.8	47.8	49.6
	$w_{cr,max}$, mm	≤2.00	≤2.00	≤2.00	≤0.50
Failure	Ω	1.9	2.5	2.5	3.7

Note: BT72-6 = 72 in. bulb tee with 0.6 in. strand; BT72-7 = 72 in. bulb tee with 0.7 in. strand; $f_{c,max}$ = maximum concrete compression stress; $f_{t,max}$ = maximum concrete tension stress; NU200-6 = 2000 mm University of Nebraska I-girder with 0.6 in. strand; NU2000-7 = 2000 mm University of Nebraska (NU) I-girder with 0.7 in. strand; $w_{cr,max}$ = maximum predicted crack width; Ω = overstrength factor (see Table 4). 1 mm = 0.0394 in.; 1 MPa = 0.145 ksi.

procedure is expected and has been identified previously.[6] The design procedure used gross section properties to assess the stress condition (that is, $P/A \pm Pe/I$, where A is the gross section area, P is the total prestressing force, and I is the gross moment of inertia of the section). Near the girder end, where the prestressing force is introduced primarily in the flange, the entire section area A was not engaged for some distance into the beam. The prestressing force near the girder end is, in effect, resisted over a smaller area, resulting in a higher stress. The spreading of the compression force over the depth of the beam can be visualized at the left end of the stress plots (Fig. 4). This effect was more pronounced for girders having large prestressing forces, and in deeper girders with uniform bottom flanges (such as BT and NU girders). The highly stressed bottom flange represents a smaller portion of the gross section area for a deeper girder, which leads to a greater discrepancy between actual local stresses and those calculated based on the gross area. This effect can be mitigated by additional debonding to introduce the prestressing force more gradually along the span.

At the Service I limit state, tensile stress was not observed to exceed 0.50 (0.19 in ksi units) and at the Service III limit state, compressive stress did not exceed $0.60f_c'$ in any case. All designs were governed by the Strength I limit state. Consistent with the design goal of maximizing girder length, all girders met, but demonstrated relatively little reserve capacity above, the Strength I limit state. As may be expected for long girders, relatively significant decompression and cracking were observed in the midspan regions, particularly for the PCI BT72 girders (Fig. 4).

In these comparisons, the number of strands for each girder type was the same and, thus, the total prestressing force was approximately 35% greater for the models having 0.7 in. (17.8 mm) strands (Table 1). Nonetheless, the spans increased only 18% and 12% for the PCI BT72 and NU2000 girders, respectively. This combination manifested as greater precompression near the midspan, resulting in a higher decompression load and less cracking. Because the span increase for the NU girder was proportionally less, the counteracting effects of the applied moment were less significant. The improved cracking behavior of NU2000-7 (2000 mm [79 in.] University of Nebraska girder with 0.7 in. diameter strand) compared with NU2000-6 (with 0.6 in. [15.2 mm] diameter strand) at Strength limit I is evident in Fig. 4.

Failure of the finite element models occurred due to an inability of the models to (mathematically) converge at loads greater than the Ω values (Table 6). This lack of convergence was associated with the relatively conservative linear bond slip model used. (This model was calibrated for an AASHTO LRFD specifications-compliant transfer length L_t of $60d_b$.) As expected, the finite element model predicted extensive cracking near the girder midspan (Fig. 4). Although the stresses in the strands were approaching rupture, none of the models predicted that the strand would rupture at the failure load attained; rather, they predicted that the degree of cracking would lead to a bond slip failure between closely spaced cracks. The 31 MPa (4.5 ksi) slab experienced considerable damage in the PCI BT72 models. A stronger and, therefore, stiffer slab would result in a relatively minor improvement in behavior. Based on the finite element model behavior reflect-

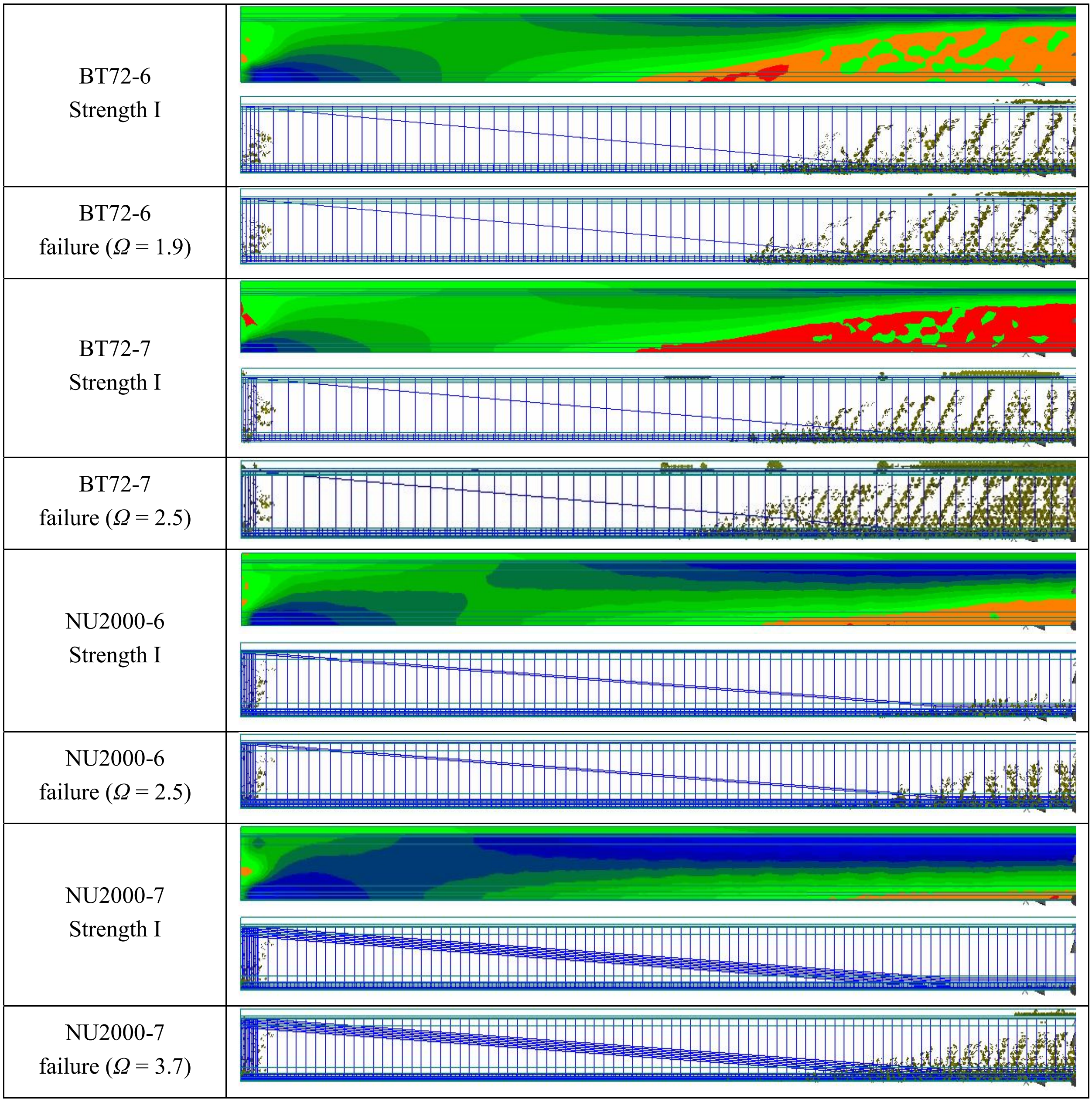


Figure 4. Finite-element-predicted longitudinal stress contours and crack patterns. A half span is shown with support at left and midspan at right. Note: BT72-6 = 72 in. bulb tee with 0.6 in. strand; BT72-7 = 72 in. bulb tee with 0.7 in. strand; NU200-6 = 2000 mm University of Nebraska I-girder with 0.6 in. strand; NU2000-7 = 2000 mm University of Nebraska (NU) I-girder with 0.7 in. strand; Ω = overstrength factor. 1 mm = 0.0394 in.; 1 in. = 25.4 mm.

ing the design objectives, the design approach for the parametric study was validated.

Effects at girder ends

The finite element models provided some insight into control of web-splitting cracks expected to occur due to the large prestressing forces near the girder ends. The models included the vertical web reinforcement, which was arranged over the initial $h/4$ length of the girder (Table 5), to resist splitting. **Figure 5** shows the associated cracking, which was expected to become more significant but was not expected to propagate along the girder at the Strength I limit state. However, the cracking extended beyond the $h/4$ distance over which the concentrated reinforcement was provided. This result supports providing the required splitting reinforcement over a longer length, as is permitted in Washington state[17] and elsewhere, or as proposed by Tuan et al.[18] The Washington State Department of Transportation limits the splitting reinforcement to pairs of no. 5 bars (16M) at 2.25 in. (57.2 mm) spacing but permits this detail

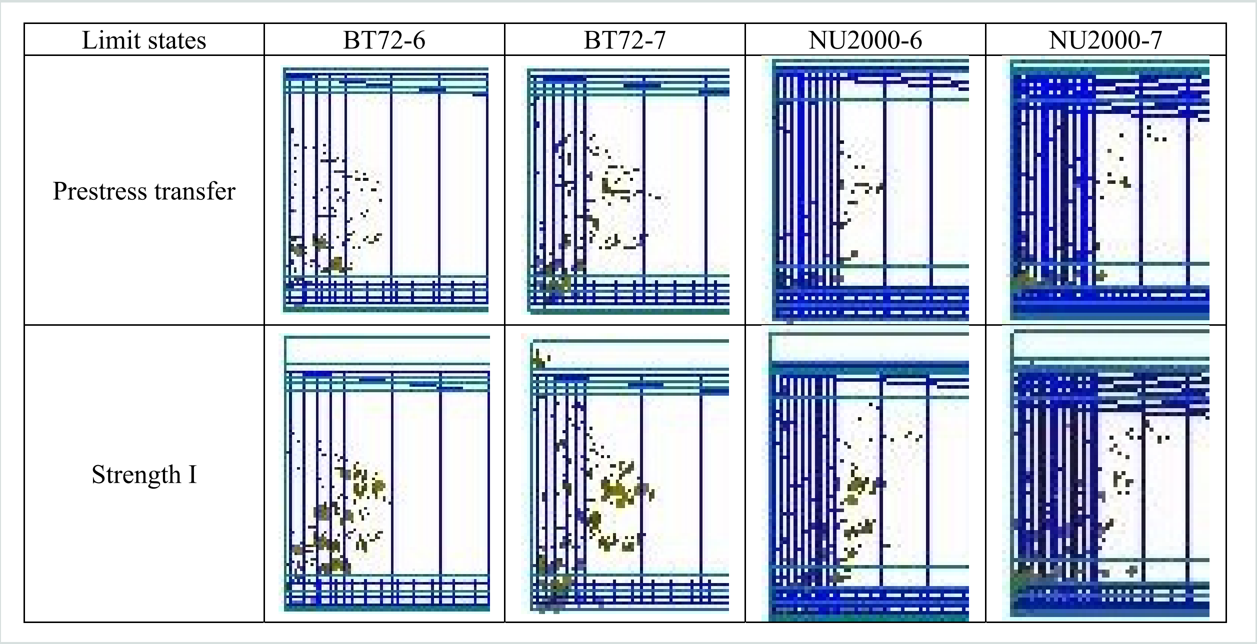


Figure 5. Finite-element-predicted crack patterns over first 1.5 m (4.9 ft) of girder ends (support at left). Note: BT72-6 = 72 in. bulb tee with 0.6 in. strand; BT72-7 = 72 in. bulb tee with 0.7 in. strand; NU200-6 = 2000 mm University of Nebraska I-girder with 0.6 in. strand; NU2000-7 = 2000 mm University of Nebraska (NU) I-girder with 0.7 in. strand. 1 mm = 0.0394 in.; 1 in. = 25.4 mm.

to extend beyond *h*/4 to accommodate all required bars. Tuan et al. proposed that one-half of required splitting reinforcement be located within *h*/8, with the remainder being extended to *h*/2.

Transverse bulb confinement reinforcement

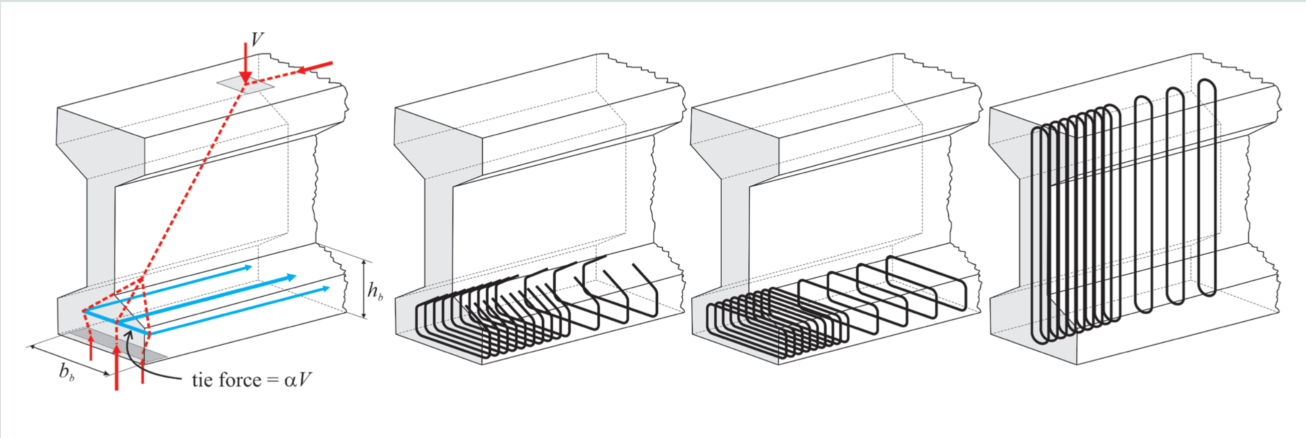
Article 5.9.4.4.2 of the AASHTO LRFD specifications requires minimum confinement of the bulb of single-web sections consisting of at least no. 3 (10M) hoops spaced at 6.0 in. (150 mm) over a length 1.5*h* from the end of the beam. For heavily loaded or long-span components (as considered herein), this requirement has been inadequate in some cases.[3,6,19] Shahrooz et al.[6] identified the development of tension oriented transversely across the bulb of single-web sections as a potential failure mode requiring tie reinforcement across the bulb width to control associated longitudinal cracking at the Strength I limit state. The magnitude of tie forces is affected by girder geometry, in particular the ratio of bearing width to flange depth b_b/h_b (**Fig. 6**). Girders with wide, flat bulbs are most susceptible to developing large tie forces. The strut-and-tie modeling approach proposed by Harries et al.[19] (Fig. 6) was used to design bulb confinement reinforcement intended to mitigate lateral splitting failures at the ultimate limit state for all 448 single-web design cases in the parametric study; details are reported in Harries et al.[19]

All 448 single-web design cases generated for the parametric study exhibited transverse tie forces that could be resisted without requiring unreasonably large tie bars or violating bar spacing requirements in most cases. Based on practical considerations regarding tie placement and concrete consolidation, ties should be no. 5 bars (16M) or smaller, and spacing should not be less than 51 mm (2.0 in.). The greatest tie-force requirement observed was 1.7 kN/m (9.8 kip/in.) predicted for an NU2000 girder with 0.7 in. (17.8 mm) strand and a span of 56.4 m (185 ft), where f_c' was 103 MPa (15 ksi) and *S* was 3.05 m (10.0 ft). This tie force could be resisted by no. 3 (10M) hoops spaced at 32 mm (1.3 in.), which violated bar-spacing requirements; no. 4 (13M) hoops spaced at 57 mm (2.2 in.); or no. 5 (16M) hoops spaced at 95 mm (3.7 in.). Though congested, all options are feasible (Fig. 6). Table 5 presents examples of the required bulb confinement reinforcement for the finite element modeled PCI BT72 and NU2000 girders.

Despite the greater total prestressing force present when 0.7 in. (17.8 mm) strands are used, the tie force is only marginally affected.[19] Using 0.7 in. strands may result in fewer strands and, thus, it is easier to use a preferential strand pattern to minimize tie forces.[19] The greater strand debonding required when larger prestressing forces are present also reduces the tie force because this reduces anchorage stresses at the girder end. On the other hand, the use of harped strands can increase tie forces because there are fewer strands in the bulb at the girder end and the harped strands are aligned with the web. Thus, debonding was given preference over harping in this study. Allowing debonding ratios greater than 0.25, as proposed by Shahrooz et al.[6] and adopted in the AASHTO LRFD specifications,[4] also affects reduced confinement-tie requirements.

Web-splitting reinforcement

Article 5.9.4.4.1 of the AASHTO LRFD specifications requires that vertical splitting reinforcement (Fig. 6) be



Strut-and-tie modeling to determine confinement tie forces

Single-leg confinement tie reinforcement

Hoop confinement reinforcement for greater tie forces

Example of splitting reinforcement (vertical hoops shown)

Figure 6. Confinement requirement for single-web girder (bulb-tee shape shown). Note: b_b = bearing width; h_b = flange depth; V = shear force; α = ratio of tie force to shear force.

provided in the girder web to resist a force equal to 4% of the total prestressing force: $0.04A_{ps}f_{pi}$ (where A_{ps} is the area of the prestressing strand). Splitting forces (and the resulting reinforcement requirement) are proportional only to prestressing force. Only debonding (not harping) can mitigate the splitting force (since only bonded strands are included in A_{ps}). Once again, allowing debonding ratios greater than 0.25 will mitigate splitting forces and the resulting reinforcement congestion.

The greatest splitting force requirement observed in the parametric study was 1.45 kN/m (8.3 kip/in.), which was observed for multiple University of Nebraska 900 mm (35 in.) (NU900) girders with 0.6 in. (15.2 mm) strands. (Harping, rather than debonding, was more effective at increasing span length for this type of section.) These cases require bundled pairs of no. 5 (16M) hoops spaced at 76 mm (3.0 in.). Congestion could be mitigated by extending the region over which splitting reinforcement was placed. When splitting reinforcement was permitted to be extended over $h/3$, rather than $h/4$, the splitting reinforcement requirement for the NU900 girders could be met using single no. 5 hoops spaced at 51 mm (2.0 in.), bundled pairs of no. 5 hoops spaced at 102 mm (4.0 in.), or pairs of no. 4 (13M) hoops spaced at 51 mm. Table 5 includes examples of the required splitting reinforcement for the finite element modeled PCI BT72 and NU2000 girders.

Long-span girder stability

This paper has shown that span lengths of existing girder shapes may, theoretically, be increased as much as 22% when 0.7 in. (17.8 mm) strand is used. As girders become longer, stability considerations during lifting and handling can begin to control aspects of design. Stability of prestressed concrete girders is considered in terms of the potential for rollover and susceptibility to excessive deformations that would cause concrete stress limits to be exceeded. Mast[20,21] noted that prestressed girders are stiff in torsion, so lateral torsional buckling is not usually a consideration. Rollover, which is the rigid body rotation of the girder, may control long girder design.[22] Bracing is a relatively straightforward way to mitigate rollover at all handling, transportation, and erection stages; this is commonly done and represents good practice. Girders are checked for stability for the processes of moving (lifting) the girders in the plant, storage (on dunnage), transportation to the site, and lifting, as well as in their final erected geometry prior to bracing. Girders are also checked for cracking and failure, which can occur due to excessive lateral deflection. In general, a factor of safety of 1.0 is used for cracking and 1.5 for failure. Girders are also checked for rollover using a factor of safety of 1.5.

In this study, investigators used the prestressed girder stability analysis approach prescribed by PCI.[23] Fundamentally, the stability analysis calculates factors of safety FS and stipulates acceptance criteria for conditions causing cracking (FS_{cr} > 1.0), failure (FS' > 1.5), and rollover (FS_{roll} > 1.5). The analyses are rigorous, considering girder geometry and material properties at each stage as well as other factors affecting stability such as camber, prestressing force, lateral wind pressure, centrifugal force during transportation, etc. The analysis for this study was done using the Girder Stability Analysis Excel spreadsheet created by PCI and revised by the research team to address several programming errors found in the original version. The revised spreadsheet was validated using the well-documented 68 m long (223 ft) WF100G described by West;[13] the validation and example calculations are presented in Alabdulkarim[24] and Shahrooz et al.[9]

Table 7. Critical cases selected for stability analyses

Case	f'_c, MPa	Girder spacing, m	$L_{0.6}$, m	$L_{0.7}$, m	$L_{0.7}/L_{0.6}$
Washington 100 in. wide-flange I-girder (WF100G)	103	3.05	51.8	63.1	1.22
Washington 74 in. wide-flange I-girder (WF74G)	124	3.05	45.8	55.2	1.21
PCI 72 in. bulb tee (BT72)	124	3.66	34.5	41.2	1.20
Ohio 72 in. wide-flange girder (WF72)	69	2.44	50.0	56.4	1.13
Florida 96 in. I-beam girder (FIB96)	124	2.44	63.1	68.0	1.13
University of Nebraska 2000 mm I-girder (NU2000)	124	1.83	59.8	67.1	1.12

Note: f'_c = concrete compressive strength; $L_{0.6}$ = maximum achievable span for pretensioned girder with 0.6 in. diameter strand; $L_{0.7}$ = maximum achievable span for pretensioned girder with 0.7 in. diameter strand. 1 in. = 25.4 mm; 1 in. = 25.4 mm; 1 m = 3.28 ft; 1 MPa = 0.145 ksi.

Stability analysis of parametric design cases

This study focused on stability of those cross-section and span combinations that had the greatest achievable increases in span length when 0.6 in. (15.2 mm) strands were replaced with 0.7 in. (17.8 mm) strands (**Table 7**). These cases potentially represented the most efficient use of 0.7 in. strands, but they also introduced the greatest potential impacts on girder stability. The study only analyzed stability of the longer 0.7 in. strand-reinforced girders.

Table 8 lists girder-dependent input parameters required for PCI stability analysis.[23] Girder unit weight was assumed to be 2400 kg/m^3 (150 lb/ft^3) in all cases. **Table 9** reports other input parameters required for each step in the stability analysis. Complete reporting of all cases presented, including sample calculations, is available in NCHRP Web-Only Document 315[9] appendix E.

In the analyses conducted, symmetric girder support was located a distance *a* from each girder end; this location was varied to maximize the calculated factors of safety. An initial assumption of *a* equal to 0.1*L* was made, and the analyses were then revised until adequate (or maximum) factors of safety were achieved. **Table 10** reports the resulting values of *a* used. The value of *a* during transportation may be limited by interaction of the vehicle and roadway geometry—specifically, the arc swept out by the overhanging end of the girder. A maximum value *a* of 6.1 m (20 ft) was used for the transportation stage in this study. When the girder was placed into its final position, the value *a* of 152 mm (6.0 in.) was selected in all analyses.

Table 10 presents the results of stability analyses in terms of the three factors of safety prescribed by PCI.[23] In a few analyses (Table 10), additional revisions to assumptions were necessary to achieve adequate factors of safety for the long spans. A complete set of sample calculations for the NU2000 case is provided in appendix E of NCHRP Web-Only Document 315.[9]

Despite the long spans, adequate stability could be achieved with all cross sections (Table 10). When the girder factor of safety for rollover FS_{roll} was less than 1.5, the girder simply requires bracing to be placed at its ends. This use of bracing should be standard practice for all such long girders, and it was required for all of the evaluated girders, except the Washington 74 in. (1880 mm) wide-flange I-girder (WF74G), when the girders were placed in their final in situ position.

The sections with the lowest ratio of weak- to strong-axis moment of inertia I_y/I_x, WF100G and Florida 96 in. I-beam girder (FIB96), failed multiple stability checks for the girder in transportation and when placed on dunnage. West[13] offers a simple remedy: increase the width of the top flange. In the WF100G and FIB96 cases, this approach worked. The top flanges of the WF100G mod and FIB96 mod sections were increased 457 and 305 mm (18.0 and 12.0 in.), respectively, to achieve stability at all construction stages. West[13] reported that increasing the top flange of a 68.0 m (223 ft)

Table 8. Parameters required for PCI stability analysis

Parameter		WF100G	WF74G	PCI BT72	WF72	FIB96	NU2000
Girder geometry	$L_{0.7}$, m	63.1	55.2	41.2	56.4	68.0	67.1
	$b_{top\ flange}$, mm	1245	1245	1067	1245	1219	1226
	$b_{bot\ flange}$, mm	975	965	660	1016	965	975
	A, mm^2	6.99×10^5	5.32×10^5	4.95×10^5	7.50×10^5	7.59×10^5	5.83×10^5
	I_x, mm^4	6.35×10^{11}	3.06×10^{11}	2.27×10^{11}	3.51×10^{11}	6.09×10^{11}	3.29×10^{11}
	I_y, mm^4	2.86×10^{10}	3.00×10^{10}	1.71×10^{10}	4.34×10^{10}	3.21×10^{10}	2.53×10^{10}
	J, mm^4	3.56×10^9	2.73×10^9	2.57×10^9	4.75×10^9	4.60×10^9	3.01×10^9
	y_{bot}, mm	1227	904	930	909	1087	907
	w, kN/m	16.45	12.53	11.65	17.68	18.64	13.74
Strands	Straight strands	46	46	32	57	66	52
	$cgs_{straight}$, mm	104	104	98	193	150	104
	Harped strands	11	11	2	2	5	8
	cgs_{harped}, mm	216 (mid) 2324 (end)	216 (mid) 2324 (end)	152 (mid) 1778 (end)	254 (mid) 1765 (end)	361 (mid) 2324 (end)	279 (mid) 1796 (end)

Source: Data from PCI (2016).

Note: Girder unit weight was assumed to be 2400 kg/m^3 (150 lb/ft^3) in all cases. A = section area; b_{bot} flange = width of bottom flange; b_{top} flange = width of top flange; BT72 = 72 in. bulb tee; cgs_{harped} = center of gravity of harped prestressing strands; $cgs_{straight}$ = center of gravity of straight prestressing strands; FIB96 = Florida 96 in. I-beam girder; I_x = moment of inertia about strong axis; I_y = moment of inertia about weak axis; J = torsional moment of inertia; $L_{0.7}$ = maximum achievable span for pretensioned girder with 0.7 in. diameter strand; NU2000 = University of Nebraska 2000 mm I-girder; w = weight per unit length of section; WF72 = Ohio 72 in. wide-flange girder; WF74G = Washington 74 in. wide-flange I-girder; WF100G = Washington 100 in. wide-flange I-girder; y_{bot} = distance from centroid to bottom of girder. 1 mm = 0.0394 in; 1 in. = 25.4 mm; 1 m = 3.28 ft; 1 kN = 0.225 kip.

long WF100G by 305 mm addressed stability issues during transportation of these particular girders. The more-slender WF100G and FIB96 girders also tended to require stiffer supports. For example, the WF100G mod girder (WF100G girder with increased top-flange width) initially failed the cracking check for the transportation stage. A value for FS_{cr} of 1 was achieved by increasing the hauling rig stiffness K_{qtrans} 43% from 9260 kN m/rad (6830 kip ft/rad) to 13,200 kN m/rad (9750 kip ft/rad). The original stiffness was selected based on recommendations in stability analysis guidelines[23] and in consultation with practicing engineers making these calculations. The authors have not investigated whether the rig stiffnesses required are achievable in practice.

The PCI BT72 section had a thinner bottom flange width b of 660 mm (26 in.) than the other sections. (All other sections had bottom flange widths of approximately 1000 mm [39 in.].) The thinner flange width resulted in a significantly lower bearing rotational stiffness K_{qseat} calculated to be 2560 kN m/rad (1890 kip ft/rad), which is approximately 30% of the bearing rotational stiffness of the wider WF100G.[25] Increasing the rotational stiffness of the PCI BT72 section by approximately 50% was required to satisfy $FS_{cr} > 1.0$ when the girder was placed on dunnage.

Conclusion

An extensive analytical study assessed the maximum achievable girder span lengths when 0.6 and 0.7 in. (15.2 and 17.8 mm) strands were used. Investigators conducted a parametric design study with 584 cases to examine the influence of girder shape and size on the potential benefits of using 0.7 in. strands. A detailed finite element evaluation of some of the longer spans achieved was also conducted. The impacts of using 0.7 in. strands on end-region detailing requirements, prestress transfer, and handling and erection stability of long-span girders were examined. Based on the results and discussion reported in this paper, we offer the following general conclusions and observations:

- The design case studies show that one-to-one replacement of 0.6 in. strands by 0.7 in. strands was not possible given the presence of many other design constraints; among those constraints, stress limits at pretensioning release are a critical concern.

Table 9. Other input parameters required for PCI stability analysis

Lifting	Rigid extension of lift device above top of girder y_{lift}	0
	Lateral tolerance of lift device from centerline of girder e_{conn}	6.4 mm
	Lateral wind force at lifting from bed or in field $w_{wind.lift}$	0.22 kN/m
Seating on dunnage	Plan dimension of the bearing $w_{brg.seat}$	$b_{bot\ flange}$ – 51 mm
	Height from roll center to bottom of girder while seated on dunnage $y_{brg.seat}$	51 mm
	Height of roll center from bearing seat $h_{roll.seat}$	51 mm
	Bearing tolerance from centerline of girder to centerline of support $e_{brg.seat}$	6.4 mm
	Bearing rotational stiffness K_{qseat}	See NCHRP report 596
	Transverse seating tolerance from level a_{seat}	0.005 m/m
	Lateral wind force $w_{wind.seat}$	0.80 kN/m
Transportation	Bunking tolerance from centerline of girder to center-line of support $e_{bunk.trans}$	25 mm
	Hauling rig stiffness K_{qtrans}	9265 kN m/rad
	Superelevation a_{trans}	0.020 m/m
	Turn radius for adverse cross slope $radius_{trans}$	36.6 m
	Hauling rig velocity in turn Vel_{trans}	16 km/h
	Height from roll center to bottom of girder during transportation $y_{seat.trans}$	305 mm
	Horizontal distance from roll axis to center of tire group $z_{max.trans}$	915 mm
	Height of roll center above roadway $h_{roll.trans}$	1219 mm
	Lateral wind force applied during transportation $w_{wind.trans}$	0.80 kN/m
Single girder on bearings	Plan dimension of the bearing $w_{brg.seat}$	$b_{bot\ flange}$ – 51 mm
	Height of bearing $h_{brg.seat}$	51 mm
	Height from roll center to bottom of girder $y_{bgr.seat} = h_{bgr.seat}/2$	25 mm
	Height of roll center from bearing seat $h_{roll.seat} = y_{brg.seat}$	25 mm
	Bearing tolerance from centerline of girder to centerline of support $e_{brg.seat}$	6.4 mm
	Bearing rotational stiffness K_{qseat}	See NCHRP report 596
	Transverse seating tolerance from level a_{seat}	0.005 m/m
	Lateral wind force applied while seated on bearings $w_{wind.seat}$	0.22 kN/m

Note: 1 mm = 0.0394 in; 1 m = 3.28 ft; 1 kN = 0.225 kip; 1 kN m = 0.738 kip ft; 1 km/h = 0.621 mph.

Table 10. Summary of stability analysis

		WF100G	WF100G modified	WF74G	PCI BT72	OHWF72	FIB96	FIB96 modified	NU2000
I_y/I_x		0.045	0.072	0.098	0.075	0.124	0.055	0.069	0.077
Added top flange width, mm		n/a	457	n/a	n/a	n/a	n/a	305	n/a
Lift from bed* $f_{peff} = 0.67f_{pu}$ $f_c = 0.83f'_c$	a, m	6.71	4.88	3.66	1.22	4.58	7.62	6.40	7.93
	FS_{cr}	1.20	1.51	1.61	1.59	1.84	1.53	1.63	1.54
	FS'	1.58	1.58	1.61	1.59	1.84	1.53	1.63	1.54
Sitting on dunnage $f_{peff} = 0.67f_{pu}$ $f_c = 0.83f'_c$	a, m	0.15	1.22	0.30	0.30	0.30	4.58	3.36	3.05
	FS_{cr}	**0.25**	1.01[†]	2.72	1.00[‡]	2.74	**0.90**	1.25	1.34
	FS'	1.55	2.41	4.24	3.66	2.81	1.97	1.97	2.34
	FS_{roll}	1.15[§]	1.64	2.09	1.80	1.91	1.52	1.52	1.52
Transportation $f_{peff} = 0.62f_{pu}$ $f_c = f'_c$	a, m	0.15	1.22	0.30	0.30	0.30	4.58	5.12	3.95
	FS_{cr}	**0.02**	1.00[‖]	2.08	1.82	2.12	**0.60**	1.01[#]	1.00
	FS'	**1.44**	2.89	3.80	7.35	2.49	1.79	2.28	2.37
	FS_{roll}	1.23[§]	2.05	2.33	3.26	1.98	1.56	1.89	1.82
Lift in field[e] $f_{peff} = 0.62f_{pu}$ $f_c = f'_c$	a, m	6.71	4.88	3.66	1.22	4.58	7.62	6.40	7.93
	FS_{cr}	1.23	1.42	1.52	1.48	1.58	1.50	1.55	1.55
	FS'	1.54	4.52	1.52	1.51	1.58	1.50	1.55	1.55
In place $f_{peff} = 0.62f_{pu}$ $f_c = f'_c$	a, m	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
	FS_{cr}	1.60	2.13	5.06	2.37	3.63	1.50	1.77	2.27
	FS'	1.86	2.34	4.95	2.76	3.20	1.45	1.69	2.08
	FS_{roll}	0.95[§]	1.12[§]	1.52	1.14[§]	1.37[§]	0.83[§]	0.93[§]	0.93[§]

Note: bold entries are below acceptance criteria. a = distance from girder end for symmetric girder support; f_c = concrete compressive strength; = concrete compressive strength f_{peff} = effective prestressing stress; f_{pu} = tensile strength of prestressing strands; FS' = factor of safety for failure; FS_{cr} = factor of safety for cracking; FS_{roll} = factor of safety for rollover; I_x = moment of inertia about strong axis; I_y = moment of inertia about weak axis; K_{qseat} = bearing rotational stiffness; K_{qtrans} = hauling rig stiffness; n/a = not applicable; $w_{wind,trans}$ = lateral wind force during transportation. 1 m = 3.28 ft.

* Wind speeds during lifts may be limited in some cases to achieve factor of safety shown.

[†] Increase K_{qseat} by 6%.

[‡] Increase K_{qseat} 50%.

[§] Bracing at girder ends required to mitigate rollover.

[‖] Increase K_{qtrans} by 43% and reduce wwind,trans by 10%.

[#] Increase K_{qtrans} by 7%.

- When designs with 0.6 in. and 0.7 in. strand were compared, up to 22% increases in girder span length were achievable in the designs with 0.7 in. strands.

- The span length of existing girder shapes optimized for 0.6 in. strands (such as NU and WF) was not appreciably increased when 0.7 in. strands were used. Less-efficient shapes (such as PCI bulb tees) exhibited greater potential increases in their spans when 0.7 in. strand was used.

- When larger-diameter strand was used to maximize pretension force, greater relief of initial stresses at the girder ends was required. When the harping of strands is not permitted (as in the case of Texas U girders) or when the degree of strand debonding is limited, the potential benefits of using the larger 0.7 in. strands cannot be fully realized.

- The increase in achievable span length associated with 0.7 in. strand was generally proportional to girder depth.

- The design approach for the parametric design case study was validated by the finite element models.

- When similar numbers of 0.6 in. and 0.7 in. strands were used, the use of 0.7 in. strands tended to require greater debonding, which had the effect of reducing the confinement tie force (assuming a favorable strand pattern was adopted). Therefore, in terms of required confinement reinforcement, there was little difference between cases with 0.6 in. and 0.7 in. strands. The required bulb confinement reinforcement was met for all the cases, and no constructability issues were anticipated.

- The increased prestressing force from the use of 0.7 in. strands resulted in greater splitting forces, leading to potentially more congested web reinforcing steel requirements at the beam ends. Nevertheless, the required reinforcement was met for all the cases considered, and no constructability issues were anticipated.

- The finite element models demonstrated that cracking extended beyond the $h/4$ distance over which concentrated splitting reinforcement was provided. This result supports designs in which the required splitting reinforcement is provided over a longer length, as is permitted by some states.

- The potential for longer achievable spans increases the susceptibility of girders to instabilities. As is required for much shorter girders than those considered here, end braces must be installed immediately upon placement on bearings to provide safety against rollover. For other conditions, the following measures can improve safety against stability effects:

  — Refining lift points and symmetric dunnage support locations a can optimize resistance to stability effects. The value of a may be practically limited during transportation based on vehicle geometry and routes chosen.

  — Increasing the width of the top flange of a girder thereby increasing I_y/I_x has a pronounced effect on improving stability.

  — Providing stiffer transportation or dunnage support—assuming that this is possible—improves stability.

  — Girders with relatively thin bottom flanges (bulb-tee sections in this study) are more susceptible to rollover while supported on dunnage or in transportation.

The parametric study was intended to be illustrative and to identify trends. Many assumptions were made, and the results presented are constrained by these.

Acknowledgments

The study reported in this paper was funded by the National Cooperative Highway Research Program (NCHRP) project 12-109. See NCHRP report 9943 and NCHRP web-only document 3159 for the complete report.

References

1. ASTM International. 2015. *Standard Specification for Low-Relaxation, Seven-Wire Steel Strand for Prestressed Concrete*. ASTM A416/ASTM A416M 15. West Conshohocken, PA: ASTM International.

2. AASHTO (American Association of Highway and Transportation Officials). 2020. *Standard Specification for Steel Strand, Low-Relaxation Uncoated Seven-Wire for Concrete Reinforcement*. AASHTO M 203M/M 203. Washington, DC: AASHTO.

3. Shahrooz, B. M., R. A. Miller, K. A. Harries, and R. Castrodale. 2022. *Use of 0.7 in. Diameter Strands in Precast Pretensioned Girders*. NCHRP report 994. Washington, DC: TRB (Transportation Research Board). https://doi.org/10.17226/26677.

4. AASHTO. 2020. *AASHTO LRFD Bridge Design Specifications*. 9th ed. Washington, DC: AASHTO.

5. Technical Committee ECISS/TC 19 SC2. 2000. *Prestressing Steels—Part 1: General Requirements*. Draft prEN 10138-1. Brussels, Belgium: European Committee for Standardization.

6. Shahrooz, B. M., R. A. Miller, K. A. Harries, Q. Yu, and H. G. Russell. 2017. *Strand Debonding in Pretensioned Girders*. NCHRP report 849. Washington, DC: TRB. https://doi.org/10.17226/24813.

7. Schuler, G. 2009. "Producer's Experience with 10,000 psi Concrete and 0.7 in. Diameter Strands." *HPC Bridge Views*, no. 54, 7–8. http://concretebridgeviews.com/2009/03/producers-experience-with-10000-psi-concrete-and-0-7-in-diameter-strands.

8. Nebraska Department of Roads. 2015. *The Roadrunner*, February/March.

9. Shahrooz, B. M., R. A. Miller, K. A. Harries, and R. Castrodale. 2022. *Details of the Study on the Use of 0.7 in. Diameter Strands in Precast Pretensioned Girders*. NCHRP Web-Only Document 315. Washington, DC: TRB. https://doi.org/10.17226/26676.

10. Concrete Technology Corp. "Alaskan Way Viaduct." Accessed February 25, 2023. http://www.concretetech.com/projects/alaskan-way-viaduct-.

11. Lallathin, T. J. 2015. “US 17-92 Interchange at SR 436.” *Aspire* 9 (4): 20–26. http://www.aspirebridge.com/magazine/2015Fall/Project-US17-92InterchangeAtSR436.pdf.

12. Canadian Precast/Prestressed Concrete Institute. 2002. “Record-Breaking Precast NU Girders Installed in Alberta.” https://www.cpci.ca/en/about_us/project_month/april_2002/.

13. West, C. 2019. “Prestressed Concrete Girders Achieve Record Lengths.” *Aspire* 13 (4): 56–57. https://www.aspirebridge.com/magazine/2019Fall/CCC-PrestressedConcreteGirders.pdf.

14. Castrodale, R. W., and C. D. White. 2004. *Extending Span Ranges of Precast Prestressed Concrete Girders*. NCHRP report 517. Washington, DC: TRB. https://doi.org/10.17226/23375.

15. Collins, M. P., and D. Mitchell. 1997. *Prestressed Concrete Structures*. New York, NY: Prentice Hall.

16. Salazar, J., H. Yousefpour, A. Katz, R. A. Abyaneh, H. S. Kim, D. Garber, T. Hrynyk, and O. Bayrak. 2017. “Benefits of Using 0.7 in. (18 mm) Diameter Strands in Precast, Pretensioned Girders: A Parametric Investigation.” *PCI Journal* 62 (6): 59–75. https://doi.org/10.15554/pcij62.6-04.

17. Khaleghi, B. 2006. “Design Memorandum: Splitting Resistance of Pretensioned Anchorage Zones.” December 23, 2006. Washington State Department of Transportation. https://wsdot.wa.gov/eesc/bridge/designmemos/11-2006.htm.

18. Tuan, C. Y., S. A. Yehia, N. Jongpitaksseel, and M. K. Tadros. 2004. “End Zone Reinforcement for Pretensioned Concrete Girders.” *PCI Journal* 49 (3): 68–82. https://doi.org/10.15554/pcij.05012004.68.82.

19. Harries, K. A., B. M. Shahrooz, B. E. Ross, P. Ball, and H. R. Hamilton. 2019. “Modeling and Detailing Pretensioned Concrete Bridge Girder End Regions Using the Strut and Tie Approach.” *Journal of Bridge Engineering* 24 (3). https://doi.org/10.1061/(ASCE)BE.1943-5592.0001354.

20. Mast, R. F. 1989. “Lateral Stability of Long Prestressed Beams Part 1.” *PCI Journal* 34 (1): 34–53. https://doi.org/10.15554/pcij.01011989.34.53.

21. Mast, R. F. 1993. “Lateral Stability of Long Prestressed Beams Part 2.” *PCI Journal* 38 (1): 70–88. https://doi.org/10.15554/pcij.01011993.70.88.

22. Zureick, A. H., L. Kahn, K. M. Will, I. Kalkan, J. Hurff, and J. H. Lee. 2009. *Stability of Precast Prestressed Concrete Bridge Girders Considering Sweep and Thermal Effects*. Atlanta, GA: Georgia Institute of Technology. https://www.dot.ga.gov/BuildSmart/ResearchDocuments/05-15.pdf.

23. PCI Bridges Committee. 2016. *Recommended Practice for Lateral Stability of Precast, Prestressed Concrete Bridge Girders*. CB-02-16. Chicago, IL: PCI. https://doi.org/10.15554/CB-02-16.

24. Alabdulkarim, A. 2021. “Use of 0.7 in. Diameter Prestressing Strand in Bridge Girders: Bond Behavior and Girder Stability.” PhD diss., University of Pittsburgh. https://d-scholarship.pitt.edu/40487.

25. Stanton, J. F., C. W. Roeder, P. MacKenzie-Helnwein, C. White, C. Kuester, and B. Craig. 2008. *Rotation Limits for Elastomeric Bearings*. NCHRP report 596. Washington, DC: TRB. https://doi.org/10.17226/23131.

Notation

a = distance from girder end for symmetric girder support

a_{seat} = transverse seating tolerance from level

a_{trans} = superelevation

A = section area

A_{ps} = area of prestressing strand

b = bottom flange width

b_b = bearing width

$b_{bot\ flange}$ = bottom flange width

$b_{top\ flange}$ = top flange width

cgs_{harped} = center of gravity of harped prestressing strand

$cgs_{straight}$ = center of gravity of straight prestressing strand

d_b = nominal diameter of prestressing strand

dr = debonding ratio

DC = weight of components (barrier walls and appurtenances)

DW = weight of wearing surface

e = distance between the centroid of the cross section and the centroid of prestressing steel

$e_{brg.seat}$ = bearing tolerance from centerline of girder to centerline of support

$e_{bunk.trans}$ = bunking tolerance from centerline of girder to centerline of support

e_{conn} = lateral tolerance of lift device from centerline of girder

E_c = modulus of elasticity of concrete corresponding to

E_{ci} = modulus of elasticity of concrete corresponding to

f_c = concrete compressive strength

f'_c = specified concrete compressive strength

f'_{ci} = concrete strength at strand release

$f_{c,max}$ = maximum concrete compressive stress

f_{peff} = effective prestressing stress

f_{pi} = initial prestressing stress

f_{pu} = tensile strength of prestressing strand

$f_{t,max}$ = maximum concrete tensile stress

FS = factor of safety

FS' = factor of safety for failure

FS_{cr} = factor of safety for cracking

FS_{roll} = factor of safety for rollover

$g_{M,int}$ = distribution factor for flexure of interior girders

h = girder depth

h_b = flange depth

$h_{brg.seat}$ = height of bearing

$h_{roll.seat}$ = height of roll center from bearing seat

$h_{roll.trans}$ = height of roll center above roadway

IM = impact factor

I_x = moment of inertia of cross section about strong axis

I_y = moment of inertia of cross section about weak axis

J = torsional moment of inertia

k_t = distance between the centroid of the cross section and the top kern point

K_{qseat} = bearing rotational stiffness

K_{qtrans} = hauling rig stiffness

L = span length

LL = HL93 live load

LL_{lane} = lane load component of HL93 live load

LL_{truck} = truck axle load component of HL93 live load

L_t = transfer length

$L_{0.6}$ = maximum achievable span for pretensioned girder with 0.6 in. (15 mm) diameter strand

$L_{0.7}$ = maximum achievable span for pretensioned girder with 0.7 in. (18 mm) diameter strand

P = total prestressing force

$radius_{trans}$ = turn radius for adverse cross slope

S = spacing

t_f = slab thickness

V = shear force

Vel_{trans} = hauling rig velocity in turn

w = girder weight per unit length

$w_{brg.seat}$ = plan dimension of the bearing

w_c = unit weight of concrete

$w_{cr,max}$ = maximum predicted crack width

$w_{wind.lift}$ = lateral wind force at lifting from bed or in field

$w_{wind.seat}$ = lateral wind force

$w_{wind.trans}$ = lateral wind force

y_{bot} = distance from centroid of section to bottom of girder

y_{lift} = rigid extension of lift device above top of girder

$y_{brg.seat}$ = height from roll center to bottom of girder

$y_{seat.trans}$ = height from roll center to bottom of girder

$z_{max.trans}$ = horizontal distance from roll axis to center of tire group

α = ratio of tie force to shear force allowing tie force to be written in terms of shear force

η = ratio of concrete compressive strength at strand release to specified concrete compressive strength

Ω = overstrength factor

About the authors

Kent A. Harries, PhD, FASCE, FACI, FIIFC, PEng, is a professor in the Department of Civil and Environmental Engineering at the University of Pittsburgh in Pittsburgh, Pa.

Bahram M. Shahrooz, PhD, is a professor of structural engineering at the University of Cincinnati in Cincinnati, Ohio.

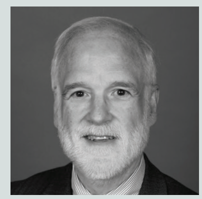
Payne D. Ball is a structural engineer with Oldcastle Infrastructure and a former graduate student at the University of Cincinnati.

TianQiao Liu, PhD, is an associate professor at Beijing University of Technology in China and a former postdoctoral researcher at the University of Pittsburgh.

Venkata S.S.P. Sathiraju is a bridge engineer with Gresham Smith and a former graduate student at the University of Cincinnati.

Abdullah Alabdulkarim, PhD, is an assistant professor at King Saud University in Riyadh, Saudi Arabia.

Richard A. Miller, PhD, is a professor of civil engineering at the University of Cincinnati.



Reid W. Castrodale is president of Castrodale Engineering Consultants in Concord, N.C.

Abstract

It has been proposed that 0.7 in. (17.8 mm) diameter prestressing strand be permitted for use in bridge girders. If 0.6 in. (15.2 mm) diameter strand is replaced on a one-to-one basis with 0.7 in. strand, the pretensioning force can be increased by 35%. When designs use 0.7 in. strands as well as high concrete strengths, longer-span prestressed concrete girders may be achieved. An extensive analytical study is presented to assess the maximum girder span lengths that can be achieved when using 0.6 and 0.7 in. strands. A parametric design study with 584 cases was conducted to examine the influence of girder shape and size on the potential benefits of using 0.7 in. strands. A detailed finite element analysis of some of the longer spans achieved was also conducted. The impacts of using 0.7 in. strands on end-region detailing requirements, prestress transfer, and handling and erection stability of long-span girders were examined. Girder span increases of up to 22% were achieved using 0.7 in. strand in place of 0.6 in. strand. The larger pretension forces affected end-region detailing and increased congestion, though all resulting requirements were constructible. The longer spans affected girder stability calculations, and some girder types required a wider top flange to meet stability-related limit states.

Keywords

End-region reinforcement, girder stability, long span, prestressed concrete girders, prestressing strand, pretensioned concrete, strand.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process. The Precast/Prestressed Concrete Institute is not responsible for statements made by authors of papers in *PCI Journal*. No payment is offered.

Publishing details

This paper appears in *PCI Journal* (ISSN 0887-9672) V. 68, No.4, July–August 2023, and can be found at https://doi.org/10.15554/pcij68.4-02. *PCI Journal* is published bimonthly by the Precast/Prestressed Concrete Institute, 8770 W. Bryn Mawr Ave., Suite 1150, Chicago, IL 60631.

Reader comments

Please address any reader comments to *PCI Journal* editor-in-chief Tom Klemens at tklemens@pci.org or Precast/Prestressed Concrete Institute, c/o *PCI Journal*, 8770 W. Bryn Mawr Ave., Suite 1150, Chicago, IL 60631.