IMPACT OF HIGH STRENGTH CONCRETE ON WSDOT PRESTRESSED CONCRETE GIRDER BRIDGES: PART I – PRETENSIONED GIRDERS

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ABSTRACT

The current use of high performance concrete (HPC) in the fabrication of prestressed concrete girders has resulted in economical bridge designs with longer spans, increased girder spacing, and shallower superstructures. HPC also improves durability, resistance to cracking, and decreases permeability and the effects of volume change due to shrinkage and creep. The use of HPC in the design of precast girder bridges has presented several new challenges, including difficulties in fabrication, shipping, and erection of long slender girders. This paper presents a parametric study that has been performed to demonstrate the effect of HPC on Washington State Department of Transportation (WSDOT) standard prestressed concrete girders. The results clearly indicate that the use of HPC, along with larger diameter strands, increase the span capability of prestressed concrete girders and, in some cases, can result in fewer girder lines or smaller, less expensive girders. A survey of WSDOT's precast girder design practices is also presented.

Keywords: Prestressed Girder, HPC, LRFD, Shipping, Span Capability

INTRODUCTION

For many years in Washington State, the preferred choice for bridge superstructure construction has been prestressed concrete girders. The inherent durability of prestressed concrete and its low initial cost made the choice easy. However, up until now, the options for increasing spans beyond the typical range for prestressed concrete girders consisted of steel girders or cast-in-place post-tensioned box girders. Steel girders are typically more expensive than precast concrete girders, require more lead-time, and require long-term maintenance for corrosion protection. Cast-in-place post-tensioned box girder construction requires complex falsework support, which can be disruptive to the environment and obstructs unlimited use of the area below the bridge. They are also more time consuming to construct than bridges using prestressed concrete girders.

The recent development of long span prestressed girders has allowed the Washington State Department of Transportation (WSDOT) and other bridge owners to solve the problem of lengthening spans with the construction material they prefer. Long span prestressed girders eliminate the need for falsework, reduce on-site construction activities and schedules, reduce environmental impacts at water crossings, and minimize hazards, delays, and inconvenience to the traveling public.

HIGH PERFORMANCE CONCRETE

Newly developed, as well as long-standing concrete additives, have been used in combinations to produce more durable, workable, and higher strength structural concrete mix designs. These high performance concrete (HPC) mixes afford designers greater latitude in the use of prestressed concrete for longer spans. In addition to improving opportunities to keep piers out of waterways and traverse the wider highways required to accommodate increasing traffic demands, HPC can also provide construction economy by reducing the size of superstructure elements or the number of required girder lines.

Since HPC is much less permeable than conventional concrete, it greatly reduces the ingress of chlorides and other contaminants that can cause accelerated corrosion of the reinforcing steel. HPC also provides improved mechanical properties that make it more resistant to traffic wear, less prone to cracking during construction and under service loads, and more manageable regarding long-term deformations such as creep and shrinkage.

HPC has recently become a standard material for the fabrication and construction of long span prestressed concrete girder bridges in Washington State. Girder strengths of 7.0 ksi (48 MPa) at prestress transfer and 9.0 ksi (62 MPa) in service are the current upper limits. Higher concrete release strengths (up to 8.5 ksi (58 MPa)) are possible if curing is extended to an every-other-day cycle. Although higher design strengths are also feasible, they are not normally necessary. While the high strength properties of HPC are the primary reason for its use in prestressed concrete girders, its improved durability is the reason for its use in the cast-in-place deck slab. Embracing the new HPC technologies has spawned design and construction innovations.

WSDOT STANDARD GIRDERS

In Washington State, the use of prestressed I-girders started in the 1950's. At that time, construction of highways and freeways was greatly accelerated under the Interstate Highway Program. The challenge was to quickly and cost effectively build grade separations at highway crossings. The economy, quality of fabrication, and ease in construction of prestressed I-girder bridges met the challenge. By the late-1950's, WSDOT had developed standard I-girder sections to facilitate economical design and construction[1]. In 1990, revisions were made to the prestressed concrete girder standards incorporating the results of research done at Washington State University on girders without end blocks[2,3]. The revised standards used thicker webs in lieu of end blocks. In 1999, long span prestressed girders commonly called "super girders" were added to the WSDOT inventory[4]. The development of the long span prestressed girders was first proposed at the 1996 annual meeting between WSDOT and the Pacific Northwest PCI producers (PNW/PCI). In 2001, a newly developed prestressed trapezoidal tub girder, commonly called "bath-tubs", was adopted. Complete descriptions of WSDOT's prestressed I-girders, trapezoidal tubs, precast slabs, and decked bulb-tees are presented in the WSDOT Bridge Design Manual[5].

Today, over 80% of new highway bridges in Washington State are prestressed I-girder bridges. The current WSDOT standard pretensioned I-girder designations are W42G, W50G, W58G, W74G, WF74G, W83G, and W95G, with span capabilities of up to 185 ft (56.4 m). The WSDOT standard I-girder sections are shown in Fig. 1, and their section properties are listed in Table 1. Fig. 1 shows the "super girder" dimensions in SI units, since they were developed as hard metric sections. The WF74G girder is a shallower version of the W83G series, with wider flanges to accommodate a greater number of prestressing strands than the W74G.

The newly developed WSDOT standard pretensioned trapezoidal tub girders span up to 140 feet (42.7 m) based on the cross section dimensions and shipping weight limitations. The trapezoidal tub cross section varies both in width and depth to accommodate the desired span length and bridge width. The variation in width is from 4.0 to 10.0 ft (1.22 to 3.05 m) and the variation in depth is from 2.50 to 5.42 ft (0.76 to 1.65 m), as shown in Fig. 2. Both the maximum span and maximum girder depth were chosen to comply with the upper limits of the approximate live load distribution equations given in the AASHTO LRFD Bridge Design Specifications[6].

STRUCTURAL EFFICIENCY OF WSDOT's PRESTRESSED I-GIRDERS

WSDOT's prestressed I-girders are among the most efficient sections used in the industry[7]. Fig. 3 compares the structural efficiency of the WSDOT standard I-girders with girders from other states and organizations using Guyon's equation[4,8] for structural efficiency. Guyon's equation is based on the cross sectional properties of the girder and is expressed as:

$$\rho = \frac{r^2}{y_t y_b} \tag{1}$$

where ρ is the efficiency factor, y_t and y_b are the distance from the center of gravity of the section to the top and bottom fibers of the girder, respectively, and r is the radius of gyration of the cross section, and is equal to $\sqrt{\frac{I}{A}}$.

An increase in the $\frac{I}{A}$ ratio will result in greater girder efficiency, which can be seen when comparing the W74G and WF74G girders in Fig. 3. This can also be said of shallower girder sections, as indicated by the dashed line in Fig. 3. Studies by Sverdrup/De Leuw for the I-15 Reconstruction Project in Salt Lake City showed that maintaining the wider flanges of the W83G series while decreasing the girder height resulted in more efficient sections than the current WSDOT standards. Figs. 4-6 show the comparative span capabilities that resulted from this study. In the future, the current WSDOT standards W42G through W58G may be phased out in favor of the WF42G through WF58G series shown in Fig 7.

WSDOT DESIGN CRITERIA AND PRACTICES

WSDOT's prestressed concrete girder bridges are designed using the current AASHTO LRFD Bridge Design Specifications and additional criteria detailed in the WSDOT Bridge Design Manual.

Applicable Limit States

The following limit states are used in the design of prestressed girders:

- Temporary Stresses
    - At transfer and stripping = 1.0 DC
    - At shipping and erection = 1.2 DC or 0.8 DC
- Service I = 1.0 DC + 1.0 DW + 1.0 (LL+IM)
    - For tension outside the longitudinal precompressed tensile zone, and compression stresses, after losses.
    - For compression stresses after losses due to live load plus one-half the sum of effective prestress and permanent loads.
- Service III = 1.0 DC + 1.0 DW +0.8 (LL+IM)
    - For tension in the longitudinal precompressed tensile zone after losses.
- Strength I = 1.25 DC + 1.5 DW +1.75 (LL+IM)
    - For ultimate flexural and shear capacity.

The limit state load modification factor η, for ductility, redundancy and operational importance, is taken as 1.0 for all prestressed girder bridges.

Loads

The vehicular live load is taken as the AASHTO LRFD HL-93 notional loading with dynamic load allowance and live load distribution factors as required by the specifications. When distribution factors are computed by the lever rule, the multiple presence factors are taken as required by the LRFD Specifications with the exception that for one lane the multiple presence factor is taken as 1.0.

Allowable Stresses

The allowable concrete stresses at lifting and shipping and at the service limit states are shown in Table 2. Current WSDOT design practice does not allow any tension in the precompressed tensile zone at the Service III limit state. Allowable stresses for prestressing strands are $0.75f_{pu}$ at transfer and $0.8f_{py}$ at the service limit state.

Prestress Losses

For ordinary designs, in lieu of more accurate loss calculations, the prestress losses for low relaxation 270 ksi (1860 MPa) strands are taken as shown in Table 3. Losses due to elastic shortening are added to the time dependent losses to obtain the total prestress losses.

The AASHTO LRFD refined loss calculation method greatly overestimates the long-term losses for modern prestressed girders and typically isn't used.

The WSDOT Modified Rate of Creep Method[9] (MRC) is used if a more precise calculation of prestress losses is desired. MRC takes into account the instantaneous and time-dependent effects of each source of loss as well as the effect of the change in section stiffness due to the composite deck.

Flexural Capacity

The calculation of the flexural capacity of prestressed girders is based on the AASHTO LRFD specification, as long as the section remains rectangular. For long span girders where flanged behavior may enter the picture, the strain compatibility approach of the PCI Bridge Design Manual[10] is used for more precise calculations of flexural capacity. In the strain compatibility approach, the stress and corresponding strain in any given layer of reinforcement may be taken from a representative stress-strain relationship. WSDOT uses the stress-strain relationship given in the PCI Bridge Design Manual. In composite construction, it is common to use concrete of different strengths in the deck and girder. In this case, an average value of β_1 is taken as:

$$\beta_{1avg} = \frac{\Sigma(f'_c A_c \beta_1)}{\Sigma(f'_c A_c)} \qquad (2)$$

where:

f'_c = specified compressive strength of concrete at 28 days, ksi

A_c = area of concrete on the flexural tension side of the member

β_1 = factor for concrete strength

Ultimate Shear Capacity

The calculated shear capacity of prestressed girders is based on the AASHTO LRFD modified compression field theory. In 1996, WSDOT performed a comprehensive study of the shear strength of prestressed girders, which resulted in fixed values for the shear design parameters β and θ[11]. The fixed shear design parameters were used to produce standardized designs for the WSDOT pretensioned I-girder series W42G, W50G, W58G and W74G. Shear design is still required for long span prestressed I-girder series WF74G, W83G and W95G, as well as all trapezoidal tubs until more field experience is acquired.

Creep of HPC

WSDOT has developed a modified creep coefficient for prestressed girders made with high performance concrete and used in standard conditions.

$$\psi(t,t_i) = \frac{3.95}{6 + f'c} Ln(t+1) \qquad (3)$$

where:

$\psi(t,t_i)$ = creep coefficient at time i

f'_c = Specified compressive strength of concrete at 28 days, ksi

t = age of concrete at the time of determination of creep effects, days

The standard conditions are defined as:

- Accelerated curing
- 6 in. (150 mm) minimum section thickness
- Relative Humidity of 75%

In determining the age of concrete at initial loading, one day of accelerated curing is taken as equivalent to seven days of moist curing. The time from stressing to prestress transfer is normally taken as 24 hours, and the time from prestress transfer until the deck reaches design strength is assumed to be 120 days.

Deflection and Camber of Prestressed Girders

The final deflection of prestressed girders is taken as the summation of the elastic deflections and deflections due to the long-term effects of time-dependent parameters at different construction stages. Fig. 8 shows the idealized deflection diagram for a composite pretensioned girder with temporary top strands. To obtain a smooth riding surface on the bridge deck, the deflection due to the weight of the slab indicated as "screed camber" is added to the profile grade elevations of the deck slab. Many measurements of actual superstructure deflections have shown that once the slab is cast, the girders tend to act as if they are locked into position[5].

The deflection of prestressed girders can also be estimated by using deflection multipliers, usually based on the creep of concrete at a certain point in time[5]. To properly use these multipliers, the upward and downward components of the initial calculated deflection should be separated in order to take into account the effects of prestress loss, which only affect the upward components. Multipliers for estimating long-term deflections of prestressed girders made with high strength concrete are given in Table 4.

Design for Continuity

WSDOT designs continuous prestressed girders for an envelope of simple span and continuous span behavior. Prestressed girders are designed for positive moments from dead and live loads as if the girders were simple spans. Deck reinforcement at intermediate piers is designed for negative moments due to continuous live and superimposed dead loads. The bare girders are required to support their self-weight, plus the weight of the cast-in-place slab, haunches, formwork, and diaphragms. By designing and constructing the bridges as both simply supported and fully continuous, the full range of structural behavior is enveloped, and connection and support details are greatly simplified. This results in economical construction and a long service life.

The connection at the intermediate piers depends on the seismic zone where the bridge is located. Seismic zones 3 and 4 are assigned to bridges in Western Washington, while seismic zone 2 is generally assigned to bridges in Eastern Washington. Fixed integral diaphragms (moment resisting) are used at the intermediate piers of continuous prestressed girder bridges located in the higher seismic zones. Hinged diaphragms at the intermediate piers are generally used for the lower seismic zones. Both integral and hinged diaphragms are semi-raised, which allows the dead load of girder, wet slab, haunches, diaphragms, and forms to be carried by the lower crossbeam, while the live and superimposed dead loads are carried by full depth crossbeam. This type of construction eliminates the need for falsework and temporary supports.

Trapezoidal tub girders are made continuous using a raised diaphragm. This type of construction requires temporary supports at the intermediate piers but is the preferred option for bridge aesthetics.

HANDLING AND SHIPPING OF LONG SPAN PRECAST HPC GIRDERS

The ability to handle and ship long prestressed concrete girders is influenced by many factors, including weight, length, height, lateral stability, and mode of transportation. The impact of these variables is discussed in Reference 4. For many years, the WSDOT Standard Specifications[12] have contained provisions for the handling and shipping prestressed concrete girders. These provisions did not contemplate the extended spans that have been made possible through the use of HPC. This section discusses modifications made to WSDOT's handling and shipping criteria in light of the use of HPC, as well as some experience gained from the first few projects. Table 5 summarizes WSDOT's current criteria for handling and shipping prestressed concrete girders.

Weight of Long Span HPC Girders

For many years, the unit weight of concrete, including reinforcement, used to calculate the weight of precast girders was 160 pcf (25.13 kN/m^3). However, it was found from measurements of actual W83G girders that the unit weight for this class of girder is closer to 165 pcf (25.92 kN/m^3). Measurements taken of the as-cast cross section dimensions were in very close agreement with the plans, indicating that no significant form spread had occurred. A spreadsheet devised to calculate actual unit weights of concrete, steel, and concrete displaced by steel found that the majority of this difference was the larger quantity of steel typical for this class of girder.

The weight of long span HPC girders is the primary factor in determining whether a girder can be shipped and how much it will cost. The comfortable net weight limitation with trucking equipment currently available in Washington State is approximately 156 kips (694 kN). Some fabricators and haulers can ship girders weighing up to 182 kips (810 kN) at a reasonable delivery rate and cost. Heavier girders (in excess of 200 kips (890 kN)) can also be shipped, but at a limited delivery rate and, possibly, at a significantly higher cost, depending on the quantity of girders and proximity of the fabrication plant to the job.

In order to provide for the most competitive bidding atmosphere, WSDOT has established the following alternative design criteria for prestressed concrete girder bridge projects:

- Prestressed concrete girders with shipping weights less than 156 kips (694 kN) are designed and detailed as conventional one-piece pretensioned girders.
- Prestressed concrete girders with shipping weights between 156 and 200 kips (694 and 890 kN) are designed and detailed for both pretensioned and post-tensioned spliced girder alternatives.
- Prestressed concrete girders with shipping weights exceeding 200 kips (890 kN) are designed and detailed as post-tensioned spliced girders. In this case, a pretensioned one-piece alternative proposed by the contractor will be considered, if it can be shown that the girders can be safely shipped and erected.

Experience has shown that providing alternate designs in the 156 to 200 kip (694 to 890 kN) weight range will lead to optimum economy for the project. The alternatives provide equal opportunity to the field of bidders, and allow for innovation in balancing the increased cost of shipping single-piece girders compared with the increased costs associated with spliced girder construction.

WSDOT Special Provisions

On projects using long heavy prestressed concrete girders, WSDOT now investigates shipping and erection during the preliminary design phase to assure that the bridge can be reasonably constructed. On some projects with restricted access, girder lengths have become an issue, resulting in a spliced girder design where a one-piece pretensioned girder could otherwise have been used. Height restrictions have generally been circumvented with alternate routes or detours. Where required, WSDOT places a special provision in the project specifications describing the findings of the preliminary investigation. This provision includes information on shipping routes, estimated permit fees, escort vehicle requirements, Washington State Patrol requirements, and estimated permit approval time.

Lateral Stability

Long span prestressed girders can become laterally unstable when handled and shipped. The analysis of lateral stability is discussed in Reference 4 and elsewhere. This section discusses what has become WSDOT's standard practice for dealing with the issue of lateral stability.

WSDOT specifies the use of temporary top strands to improve the stability of long slender girders during handling and shipping. These strands are either pretensioned along with the permanent strands or are post-tensioned sometime after the forms are stripped. The choice of pretensioning or post-tensioning is left to the manufacturer depending on the production scheme to be used.

Pretensioned temporary strands are bonded within the end 10 ft (3.05 m) of the girder only, and are unbonded throughout the remainder of the girder length. Post-tensioned temporary strands are anchored with monostrand anchor plates at one end, are bonded within 10 ft (3.05m) of the other end, and are unbonded elsewhere. Block outs are provided on top of the top flange to allow access for cutting the strands once the girders are erected and stabilized. A schematic is shown in Fig 9.

Failure to release the temporary prestress force may have adverse effects on the structural behavior of the girder. WSDOT requires all temporary strands to be visibly flagged before the girders are shipped to the job site, and the bridge plans give instructions for releasing the temporary strands. The monostrand ducts used for the temporary strands are oversized and sealed to prevent binding or bonding of the strands when cut. Measurements taken of strand retraction after cutting indicate that the system allows the strands to fully relax after release.

The introduction of temporary strands to the top flange also has beneficial effects on the design of prestressed girders. The temporary top strands reduce the instantaneous deflection and long-term camber, which results in a reduction of the volume of concrete required for the cast-in-place deck haunches. This translates into less deck concrete and lower dead load moments. Also, the temporary reduction in the eccentricity of the total prestress reduces the compressive stresses in the girder at release and consequently reduces the required concrete release strength.

When pretensioned temporary top strands are used or when the strands are post-tensioned shortly after release, the effect of the strands on the long-term camber should be considered in the design. Most HPC girders can be stripped from the forms without temporary strands, at the expense of higher release strength. However, many HPC girders will require temporary strands for shipping. In cases where the temporary top strands are not considered in the design and where their effects on long-term camber would be detrimental to construction, these strands can be post-tensioned shortly before shipping, thus minimizing their effects on camber.

Handling of Trapezoidal Tub Girders

The trapezoidal tub, because of its large width, does not have a tendency to roll. The shape of the cross section provides a large moment of inertia about its vertical axis. Additionally, it can be lifted with four pick points, so the beam's self weight tends to resist end rotation. Consequently, these beams are inherently stable, and do not require the measures taken with long I-girders.

PARAMETRIC STUDY

The focus of the parametric study was to investigate the impact HPC has on the structural efficiency of WSDOT's pretensioned girders. In this study, structural efficiency manifested itself through increased span lengths, increased girder spacing, and shallower girder depths. The viability of WSDOT's HPC pretensioned girders is typically controlled by either the required concrete strength at transfer, or the shipping weights. Both of these variables are referenced as limiting factors throughout the parametric study.

Design Criteria for Parametric Study

The parametric study was based on the AASHTO LRFD Bridge Design Specifications, 2nd Edition, 1998 with interims through 2002, and the WSDOT Bridge Design Manual, July 2002. The limit state load modifiers for ductility, operational importance and redundancy were taken as 1.0. The applicable limit states for the parametric study are shown in Table 6.

For each of the cross sections, the span capability and girder spacing were determined for both normal and high strength concrete. Concrete strengths at transfer were taken as 5.5 ksi (38 MPa) for normal strength concrete and 7.5 ksi (52 MPa) for high strength concrete. Low relaxation prestressing strands of 0.5 in. (12.7 mm) and 0.6 in. (15.2 mm) diameter at 2 in. (50 mm) spacing were used for normal and high strength concrete, respectively. Dead loads were assumed to be the weight of the girder, deck and deck haunches, and standard concrete diaphragms at 40 ft (12.2 m) on center maximum. An additional load of 0.15 kip/ft (2.19 kN/m) was applied to the composite section to account for barriers or other miscellaneous superimposed dead loads.

Class 4000D (28D) concrete with a specified 28-day compressive strength of 4.0 ksi (28 MPa) is typically used for all WSDOT bridge decks. The minimum slab thickness is 7.5 in. (190 mm) with an increase up to 9 in. (230 mm) depending on the bridge configuration. A constant deck thickness of 7.5 in (190 mm) was used for the parametric study.

The creep coefficient was calculated in accordance with the WSDOT creep equation for normal exposure conditions and a relative humidity of 75%. For creep coefficient calculations, the time from stressing to prestress transfer was assumed to be 24 hours, and the time from prestress transfer until slab casting was taken as 120 days.

Prestress losses were calculated in accordance with the AASHTO LRFD Lump Sum Method. Others have shown that the AASHTO LRFD refined method for loss prediction results in a gross overestimation of losses and therefore artificially limit span capabilities (4). Prestress losses for shipping calculations were taken as 75% of the final losses. All other design criteria were according to WSDOT standard practice as described earlier.

RESULTS OF PARAMETRIC STUDY

The key material components necessary to maximize the structural efficiency of long span prestressed girders are high strength concrete and large diameter prestressing strands.

Another key component in increasing span capability is the use of temporary top prestressing strands, as discussed earlier in this paper. Table 7 summarizes the span capability of WSDOT pretensioned girders for both normal and high strength concretes. In some cases, the span capability was controlled by an allowable shipping weight of 200 kips (890 kN). In fact, this condition dominated the span capability of the W95G girders. These sections are used primarily for post-tensioned, segmental construction, or where economy can be realized by using fewer girders at a wider spacing in pretensioned applications.

Due to varying dead load demands on superstructure elements, the values in Table 7 were rounded down to the nearest 5 ft. This table is included in WSDOT's Bridge Design Manual as an aid for preliminary girder type and size evaluation.

Effect of High Strength Concrete on Span Capability

HPC allows longer span lengths using standard cross sections. An increase in the concrete strength at transfer from 5.5 to 7.5 ksi (38 to 52 MPa), along with the use of 0.6 in. (15.2 mm) diameter strands, allowed an increase in span length of approximately 20% for all WSDOT pretensioned I-girder series. Fig. 10 shows the effects of the strength of concrete at transfer on the span capability of WSDOT prestressed I-girders with no temporary top strands. The data clearly demonstrates that span capability increases with the attainable concrete strength at the release of prestress. Note that the span capability of the W95G girder is limited due to the restriction on the maximum shipping weight of 200 kips (890 kN).

Effect of High Strength Concrete on Girder Spacing

The concrete strength at transfer has a dramatic effect on potential girder spacing. Fig. 11 shows the effect of concrete strength at transfer on the spacing of WF74G girders. For this comparison, 0.6 in. (15.2 mm) diameter strands were used. For a span length of 140 feet (42.7 m), the girder spacing can be increased from 6 feet (1.8 m) to 8 feet (2.4 m). A practical implication of this is that, for a 55 ft (16.8 m) wide slab with 3.5 ft (1.0 m) overhangs, two lines of girders can be eliminated from the bridge.

Effect of High Strength Concrete on Girder Depth

The ability to use shallow prestressed girders is becoming increasingly important where vertical clearance is a design constraint. Fig. 12 shows the effects of HPC on girder depth. For a span length of 115 ft (35.0 m), a W74G girder made with normal strength concrete may be replaced with a W58G made with high strength concrete. In addition to gaining 16 inches

(405 mm) of vertical clearance, using the smaller girder section provides savings in the cost of fabrication and shipping. Where vertical clearance is not an issue, using a shallower girder sections can reduce the size of approach fills and their associated costs.

In advocating shallower girders and wider girder spacing, there may be some concern regarding time-dependent deflections and camber. The use of shallower sections may require in-depth calculations of time-dependent parameters and effective detailing to reduce creep and shrinkage. Bridge designers may wish to incorporate top temporary strands or other design features to mitigate time-dependent effects. Another option is to specify high performance concrete with favorable time-dependent properties.

Effect of Prestressing Strand Size on Span Capability

The effect of prestressing strand size on span capability is simply this: larger diameter strands can introduce more prestressing force into a given girder section. Many standard girder cross sections, particularly the bottom flange where the straight strands reside, were sized for 0.5 in. (12.7 mm) diameter strands and normal strength concrete. In order to take advantage of high strength concrete, more prestressing force must be introduced into areas that were once fully occupied by 0.5 in. (12.7 mm) diameter strands. 0.6 in. (15.2 mm) diameter strands allow for the introduction of this additional prestressing within the same cross section. Fig. 13 compares the span capability of WSDOT's pretensioned I-girders using 0.5 in. (12.7 mm) and 0.6 in. (15.2 mm) diameter strands.

Effect of Flange Width on Span Capability

Fig.14 compares the span capabilities of WF74G and W74G girders using 0.6 in. (15.2 mm) strands. For a concrete strength at transfer of 6.5 ksi (44.8 MPa) and a girder spacing of 8 ft (2.4 m), the WF74G can span 145 ft (44.2 m) while the W74G is limited to 125 ft (38.1 m). The cross section depths are approximately the same (72.83 inches (1850 mm) for WF74G and 73.50 inches (1867 mm) for W74G), while the WF74G has a larger bottom flange that can accommodate up to 20 more strands than the W74G.

Effect of Temporary Strands on Span Capability

As mentioned earlier, temporary top strands are used primarily to improve the lateral stability of long prestressed girders during handling and shipping. These strands also have beneficial effects on the in-service design of prestressed girders, though these are generally limited to a reduction of the dead load in the girder haunches. Figure 15 shows that the effects of temporary top strands on the span capability of WSDOT I-girders is relatively mild. However, the real benefit of the use of temporary top strands lies in the fact that girder configurations that would normally be outside the allowable range of concrete release strength or stability factors-of-safety can now be reasonably fabricated and shipped.

DESIGN TOOLS

To facilitate the rapid design of prestressed girders in accordance with AASHTO LRFD and WSDOT criteria, the WSDOT Bridge and Structures Office has developed design aids and computer software tools.

Design Aids

One of the first steps in the preliminary design of a prestressed girder structure is to determine the span configuration, girder size, girder spacing, and level of prestressing. With so many potential combinations, this can be a challenging task. To arrive at an efficient bridge configuration, WSDOT publishes span capability charts in its Bridge Design Manual. Using the span capability charts, designers can quickly compare design alternatives and choose a suitable bridge configuration.

Computer Software Tools

WSDOT publishes several bridge engineering computer software titles. The most popular title, PGSuper™, is used to design and perform specification compliance checking for prestressed girders. The flexural design feature determines the number and configuration of prestressing strands and required concrete compressive strengths. The specification-checking feature evaluates girders for compliance with strength, service, and detailing criteria in accordance with AASHTO LRFD specifications and the WSDOT Bridge Design Manual. Girders are also evaluated for overstress and instability during handling and transportation.

To facilitate the design of continuity reinforcement in deck slabs, the QConBridge™ program can be used to determine negative moments due to live load and superimposed dead load.

WSDOT publishes these software tools as part of its open source software effort, the Alternate Route Project. This software can be freely downloaded from the WSDOT web site at http://www.wsdot.wa.gov/eesc/bridge.

GENERAL OBSERVATIONS

The outcome of the parametric study shows that the use of high performance concrete, in conjunction with larger diameter prestressing strands, can increase span lengths well beyond the limits currently used in highway bridges. However, the challenges of handling and shipping long span prestressed girders can impose a ceiling on the span capability. These challenges can be overcome to some degree by introducing temporary prestressing strands into the top flange of the girder. In cases where shipping and handling limit span capability, the option of segmental construction is always available.

High performance concrete can result in considerable economies by allowing wider girder spacing and shallower girder cross sections. With wider girder spacing, fewer girders need to be fabricated, transported, and erected. Shallower cross sections can be extremely important when vertical clearance is a design constraint. The desired vertical clearance can be achieved without increasing the size of the approach fills and abutments.

CURRENT PROJECTS

Table 8 lists several recently completed or in-progress projects in Washington State using HPC and WSDOT's new pretensioned "super girders". The Allen Street Bridge, which was the first project to use the pretensioned "super girders", is featured in the Winter 2002 issue of ASCENT Magazine. Two long-span girders from the La Center Bridge were instrumented and monitored for prestress losses as part of NCHRP Project 18-07, "Prestress Losses in Pretensioned High-Strength Concrete Bridge Girders". The final report on this project is due in the fall of 2002, with a PCI Journal article to follow shortly thereafter.

The Methow River Bridge, which is WSDOT's first use of pretensioned "super girders", is currently under construction, and includes the largest pretensioned girders ever fabricated and shipped in the state to date. As of August 2002, Stage 1 construction is complete and open to traffic, the existing bridge has been demolished, and Stage 2 construction is underway.

The girders for the Padden Parkway Pedestrian Bridge will be even larger than the Methow girders. This bridge consists of two girders in each of three spans over I-205 and associated ramps in Vancouver, Washington. The walkway deck is 16 ft wide. For erection, the freeway must be entirely shut down since, for two of the spans, the delivery vehicles must travel southbound on the northbound lanes and ramps in order to position the large girders for the crane picks. Erection will be supported by delivering two girders for each span on three consecutive days over a single weekend.

CONCLUDING REMARKS

The availability of high strength concrete enables WSDOT engineers to design bridges with longer span lengths, fewer girder lines, and shallower girder sections, depending on the parameters of a particular project. Longer spans permit the use of fewer supports, which reduces environmental impacts at water crossings and improves traffic safety, especially at locations with high traffic congestion. Fewer girders resulting from increased girder spacing reduce fabrication, transportation, and erection costs. Shallower girders made possible by higher strength concrete create economies in the construction of approach embankments and abutments as well as improving vertical clearance.

ACKNOWLEDGEMENTS

The authors wish to thank Paul Bott of Jacobs Civil Inc. for supplying the results of the Sverdrup/DeLeuw study on the I-15 reconstruction project. The opinions and conclusions expressed in this paper are those of the authors and are not necessarily those of the Washington State Department of Transportation.

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Table 1: Section Properties of WSDOT Pretensioned I-Girders

Girder Series	Depth (in)	Area (in^2)	Center of Gravity to Bottom (in)	Moment of Inertia (in^4)	Girder Unit Weight *(k/ft)	Guyon Girder Efficiency
W42G	42.00	374	18.89	76437	0.416	0.47
W50G	50.00	526	22.77	165462	0.585	0.51
W58G	58.00	604	27.96	265374	0.672	0.52
W74G	73.50	747	38.03	547533	0.830	0.54
WF74G	72.83	912	34.92	703402	1.045	0.58
W83G	82.68	972	39.66	956329	1.114	0.58
W95G	94.50	1044	45.38	1322223	1.196	0.57

* The unit weight of girder is based on a unit weight of concrete, including reinforcement, of 160 pcf for W42G – W74G, and 165 pcf for WF74G – W95G.

1 in = 25.4 mm

1 k/ft = 14.59 kN/m

Table 2: Allowable Concrete Stresses

Loading Stages	Compression	Tension
Temporary stresses at transfer, lifting and shipping	$f_c = 0.6\,f'_c$	$f_t = 0.0948\sqrt{f'c}$* $f_t = 0.237\sqrt{f'c}$** f'_c in KSI
Final stresses at Service	Service I $f_c = 0.45\,f'_c$ due to P/S + DL $f_c = 0.60\,f'_c$ due to all loads $f_c = 0.40\,f'_c$ due to LL+IM+0.5(DL+P/S)	Service I or III f_t =0.0

* Without bonded mild reinforcement.

** With bonded mild reinforcement sufficient to resist the total tension (at 30 ksi maximum steel stress) calculated on the basis of an uncracked section.

1 ksi = 6.89 Mpa

Table 3: Time-Dependent Prestress Losses for Composite Construction

	Type of Cross Section	
Stage	**Prestressed I-Girder**	**Prestressed Tub-Girder**
Total Lump Sum Losses at: **Transfer (KSI)**	20	15
Time-Dependent losses at: **Final (KSI)**	$33.0[1.0 - 0.15\frac{f'c - 6}{6}]$	21

1 ksi = 6.89 Mpa

Table 4: Deflection and Camber Multipliers for Pretensioned Girders

Deflection at Erection Due to:	**Non-Composite**	**Composite**
Weight of Girder (Downward)	1.75	
Prestressing (Upward)	1.70	
Deflection at Final Due to:	**Non-Composite**	**Composite**
Weight of Girder (Downward)	2.50	2.20
Prestressing (Upward)	2.25	2.10
Weight of Slab (Downward)	2.30	2.15
Super Imposed Dead Loads (Downward)	2.75	2.75

Table 5: WSDOT Design Parameters for Shipping and Handling of Pretensioned Girders

	Lifting from Casting Bed	Shipping
Factor of Safety Against Cracking	1.0	1.0
Factor of Safety Against Failure	1.5	-
Factor of Safety Against Rollover	-	1.5
Impact Factors (Upward and Downward)	-	0.8 or 1.2*
Roll stiffness of trailer	-	32000 $^{in\text{-}k}/_{rad}$ for W < 164 k 40000 $^{in\text{-}k}/_{rad}$ for 164 < W < 182 k 48000 $^{in\text{-}k}/_{rad}$ for 182 < W < 200 k
Max. superelevation		6%
Girder sweep tolerance	$^{1}/_{16}$ in. per 10 ft	$^{1}/_{8}$ in. per 10 ft
Lifting Device or Truck Support Lateral Tolerance	0.25 in.	1.00 in.

* Impact Factors are not applied in the analysis of stresses in a tilted beam.

1 kips = 4.448 kN

1 ft = 0.3048 m

1 in. = 25.4 mm

1 in-k/rad = 0.113 kN-m/rad

Table 6: Design Criteria for the Parametric Study

Stages	Load cases	Limit States
Casting Yard	Self weight of: Girder	Service I
Bridge Site Stage 1	Self weight of: Girder Diaphragms Slab	Service I
Bridge Site Stage 2	Self weight of: Girder Diaphragms, Slab Traffic barriers*	Service I
Bridge Site Stage 3	Self weight of: Girder Diaphragms Slab Traffic barriers* HL-93 Live Load	Service I Service III Strength I

* Weight of traffic barriers and utilities are distributed over a maximum of 3 girders.

Table 7: Span Capability of WSDOT Pretensioned I-Girders

		f'_{ci} = 5.5 ksi, 0.5" Diameter Strands	f'_{ci} = 7.5 ksi, 0.6" Diameter Strands
Girder Type	Girder Spacing (ft)	Span Length (ft)	Span Length (ft)
W42G	6.0	75	85
	8.0	65	80
	10.0	60	70
	12.0	50	65
W50G	6.0	105	110
	8.0	90	95
	10.0	80	90
	12.0	70	80
W58G	6.0	115	125
	8.0	100	115
	10.0	95	105
	12.0	85	90
W74G	6.0	135	150
	8.0	120	140
	10.0	110	130
	12.0	100	115
WF74G	6.0	155	165
	8.0	145	155
	10.0	135	150
	12.0	120	140
W83G	6.0	165	179*
	8.0	155	175
	10.0	145	165
	12.0	135	150
W95G	6.0	167*	167*
	8.0	165	167*
	10.0	150	167*
	12.0	140	160

* The span capability is controlled by a maximum shipping weight of 200 kips.

1 ksi = 6.89 MPa

1 ft = 0.3048 m

Table 8: Recent Pretensioned HPC "Super Girder" Projects in Washington State

Bridge	Location	Girder Type	Number of Spans	Max. Girder Length (ft)	Girder Spacing (ft)	Max. # of 0.6" ϕ Strands*	Max. f'_{ci} (ksi)	Max. f'_c (ksi)	Construction Status
Nisqually Road SW Bridge	Pierce County	W83G	1	120.71	10.50	42	6.0	8.0	In Progress
Padden Pedestrian Bridge	Clark County	W83G	3	185.23	8.00	64	8.1	9.2	In Progress
Cedar Mountain Bridge	King County	W83G	3	160.50	9.00	60	7.3	8.7	In Progress
Anderson Creek Bridge	Kitsap County	W83G	1	124.47	10.00	41	5.3	6.0	In Progress
Methow River Bridge	Twisp	W83G	2	176.84	6.07	68	8.3	10.0	In Progress
La Center Bridge	Clark County	W83G	4	162.56	7.17	64	7.8	10.0	Completed
Allen Street Bridge	Kelso	W83G	7	164.58	8.5	58	6.8	7.5	Completed

- Does not include temporary top strands, if applicable.

1 ksi = 6.89 MPa

1 ft = 0.3048 m

1 in = 25.4 mm

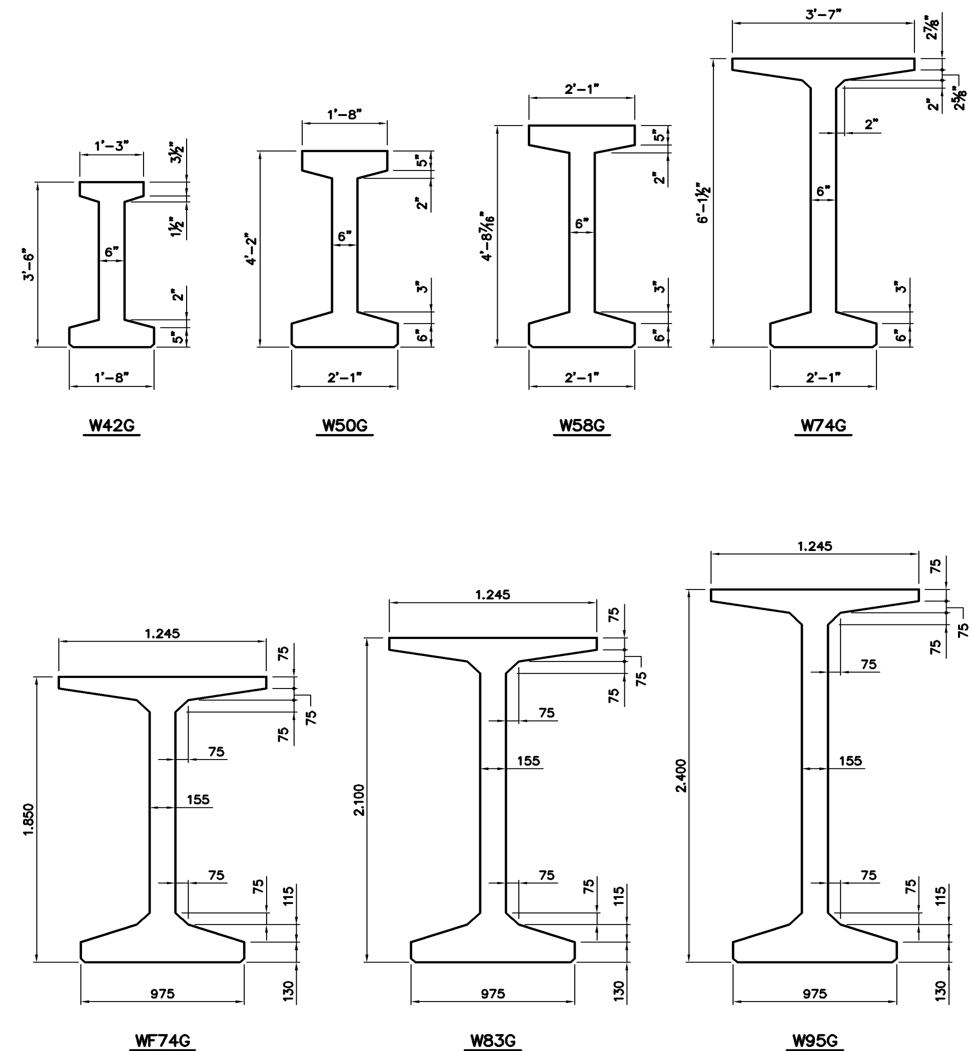

Fig. 1. WSDOT Standard Pretensioned I-Girders
1 in = 25.4 mm

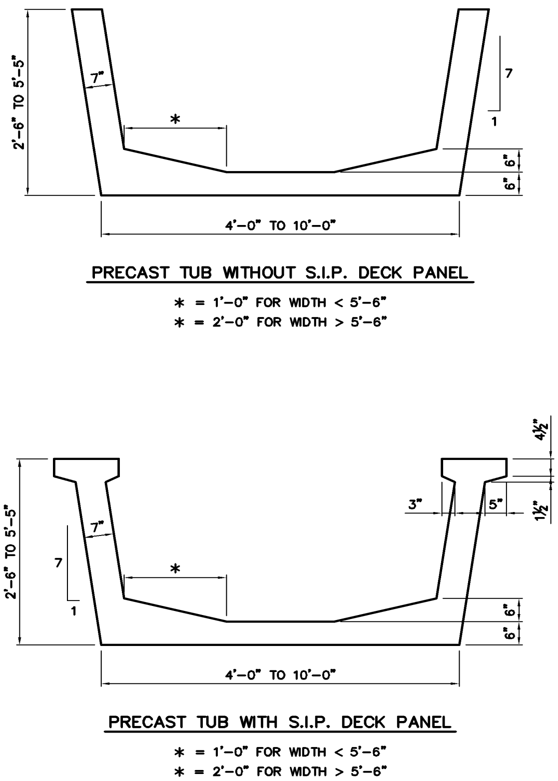

Fig. 2. WSDOT Standard Pretensioned Trapezoidal Tub Girders

1 ft = 0.3048 m

1 in = 25.4 mm

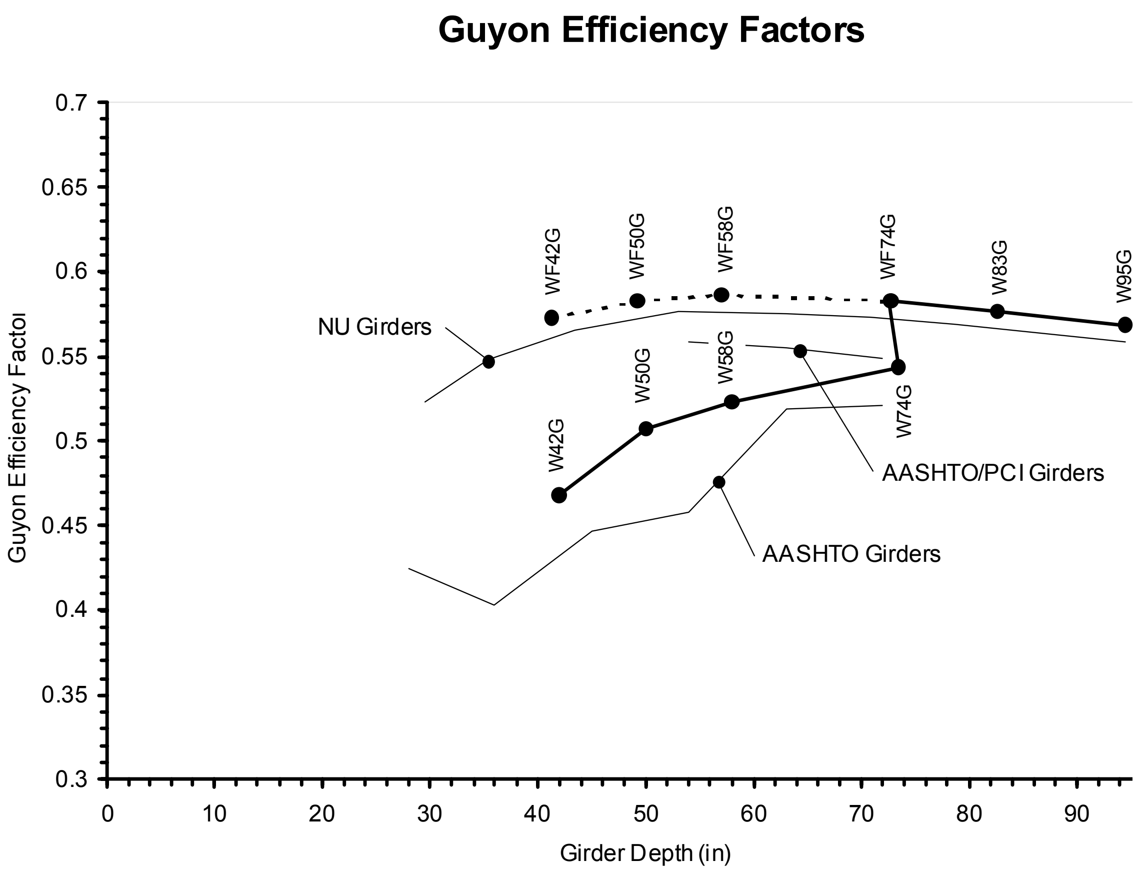

Fig. 3. Guyon Efficiency Factors

1 ft = 0.3048 m

1 in = 25.4 mm

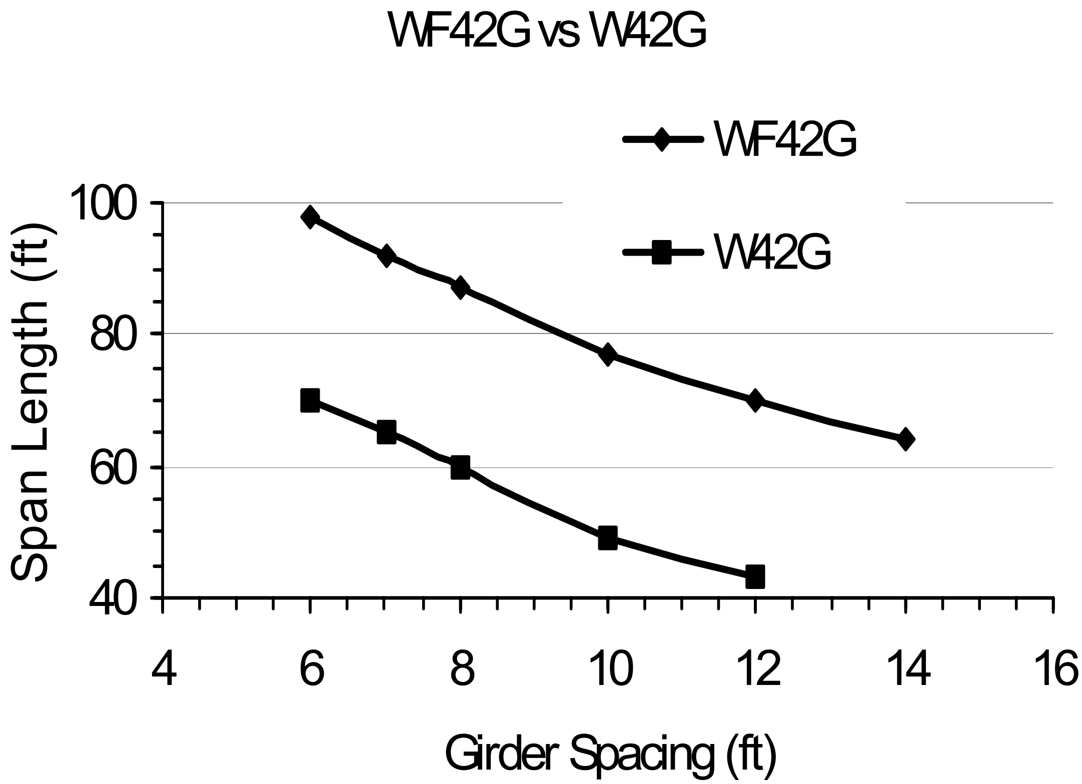

Fig. 4. WF42G vs W42G Span Capability Comparison

1 ft = 0.3048 m

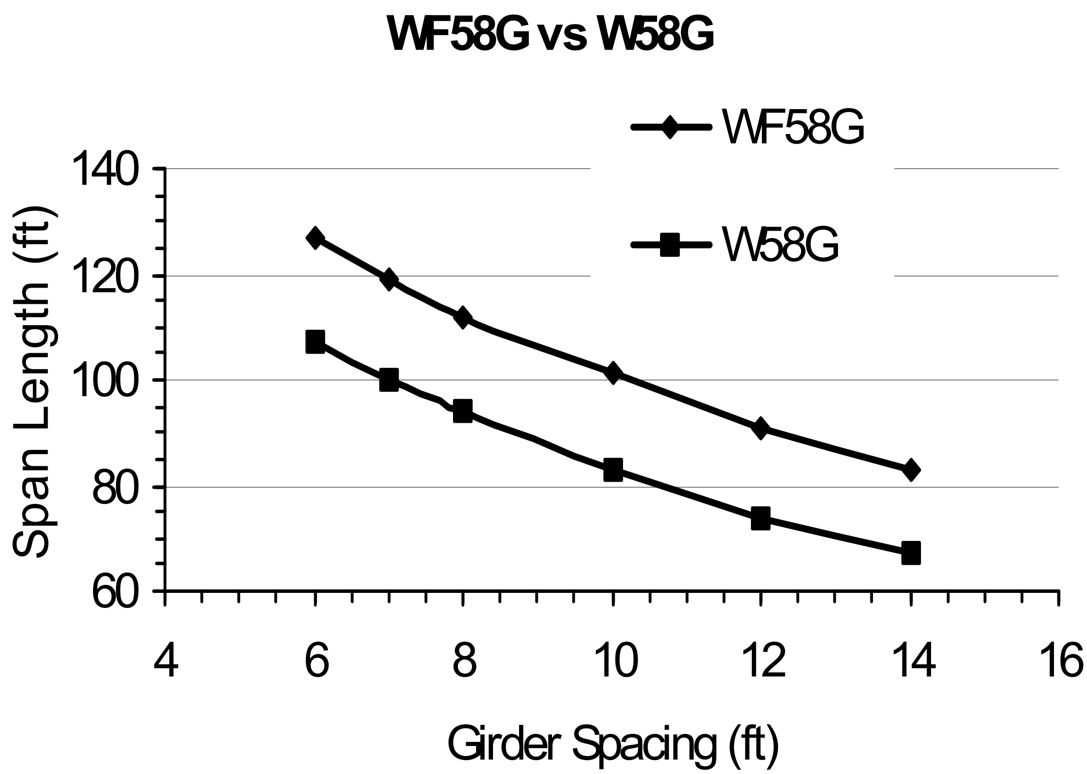

Fig. 5. WF58G vs W58G Span Capability Comparison

1 ft = 0.3048 m

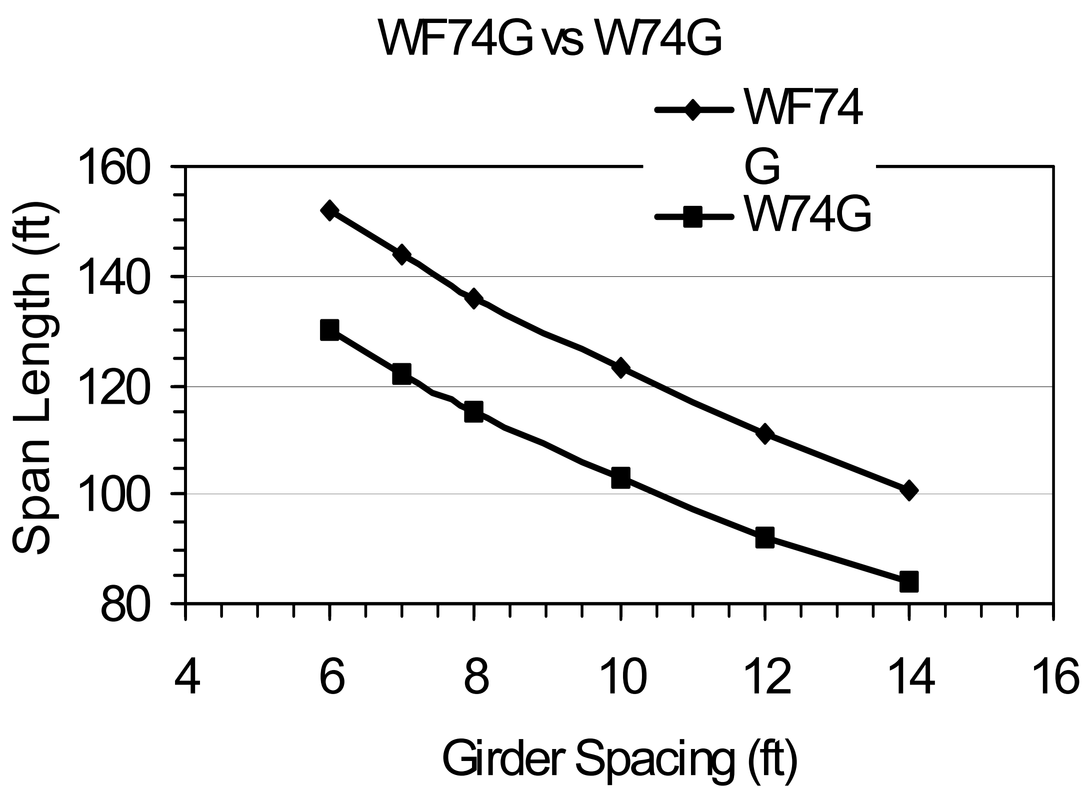

Fig. 6. WF74G vs W74G Span Capability Comparison

1 ft = 0.3048 m

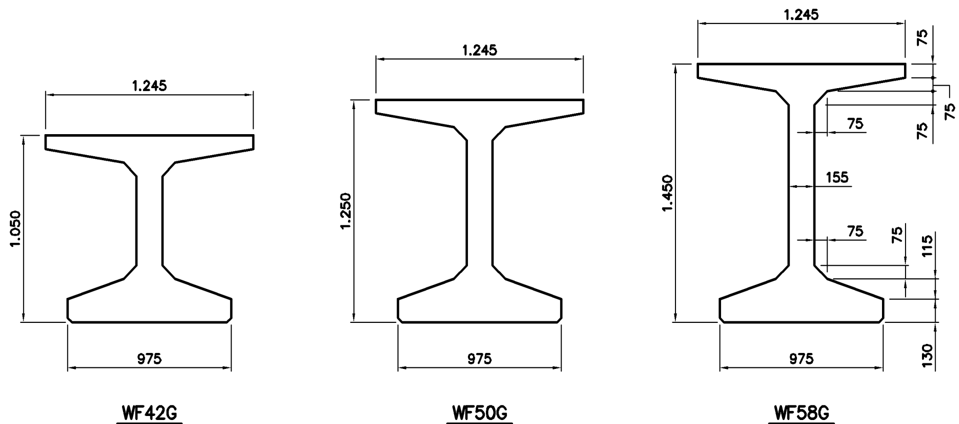

Fig. 7. Possible Future WSDOT Standard I-Girder Sections

1 mm = 0.0394 in
1 m = 39.4 in
1 m = 3.28 ft

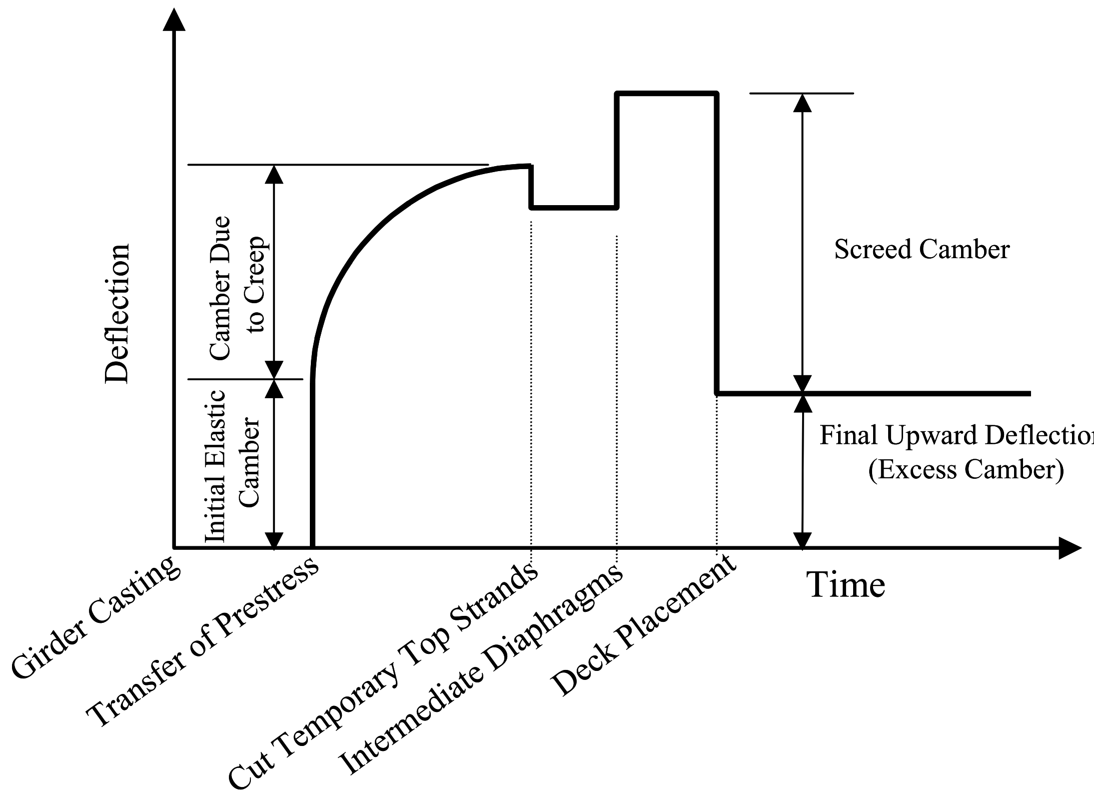

FIG. 8. Idealized Deflection Diagram for WSDOT Pretensioned Girders

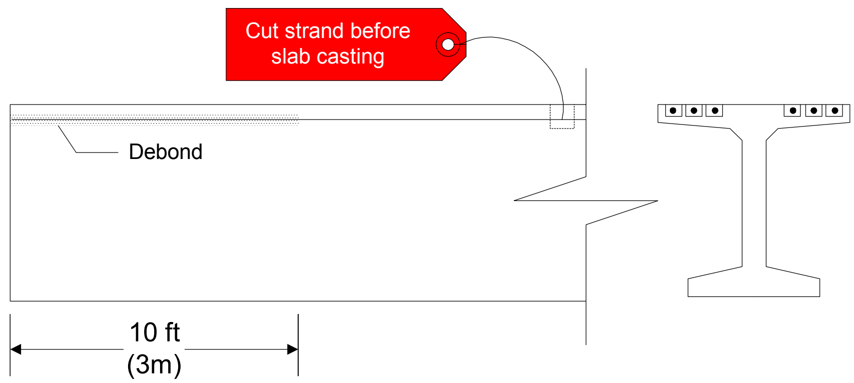

FIG. 9. Schematic of temporary strand placement and flagging

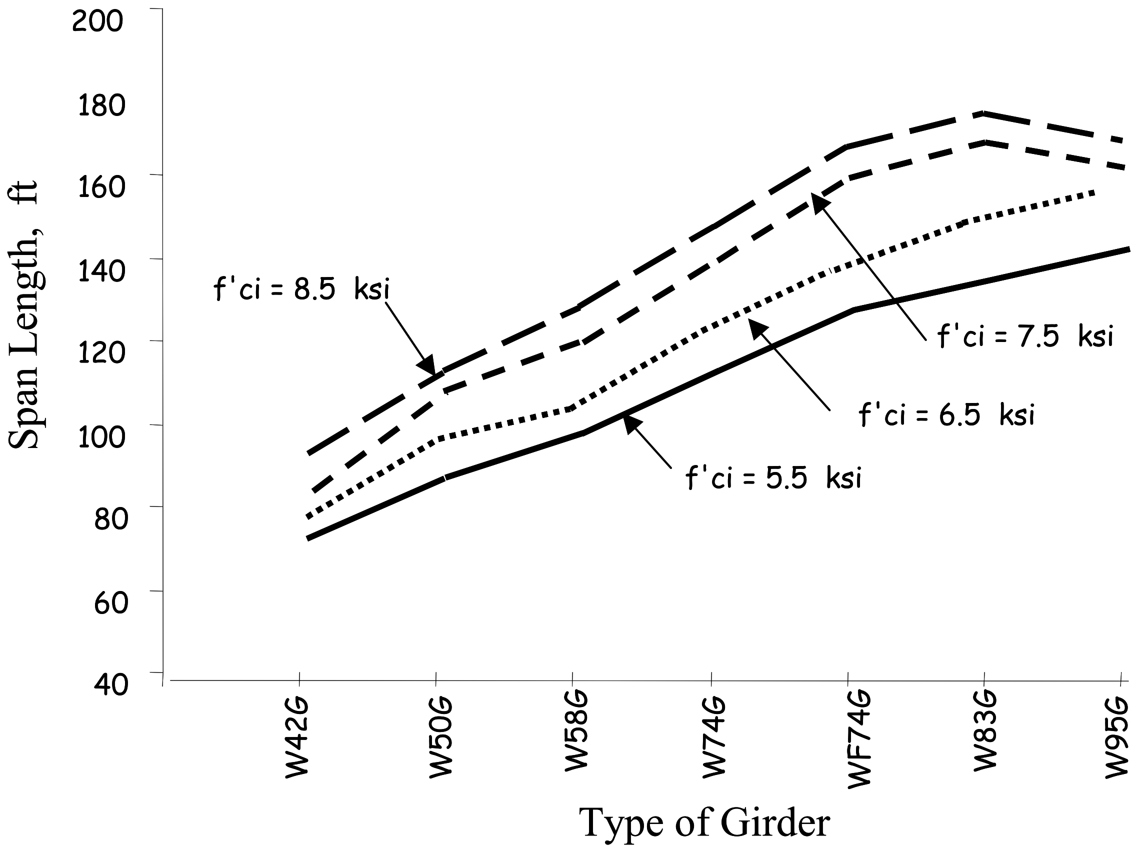

FIG. 10. Effect of Concrete Strength at Transfer on Span Capability of WSDOT Pretensioned I-Girders (0.6” f Strands)

1 ksi = 6.89 MPa

1 ft = 0.3048 m

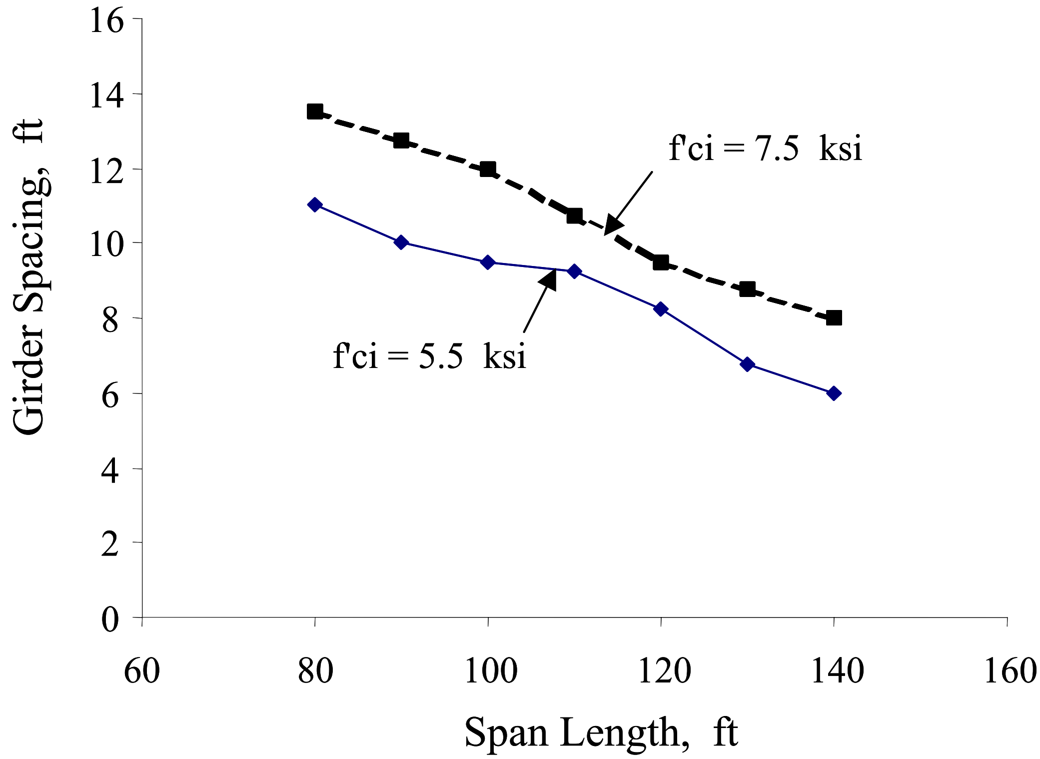

FIG. 11. Effect of Concrete Strength at Transfer on Spacing of WF74G Girders

1 ksi = 6.89 MPa

1 ft = 0.3048 m

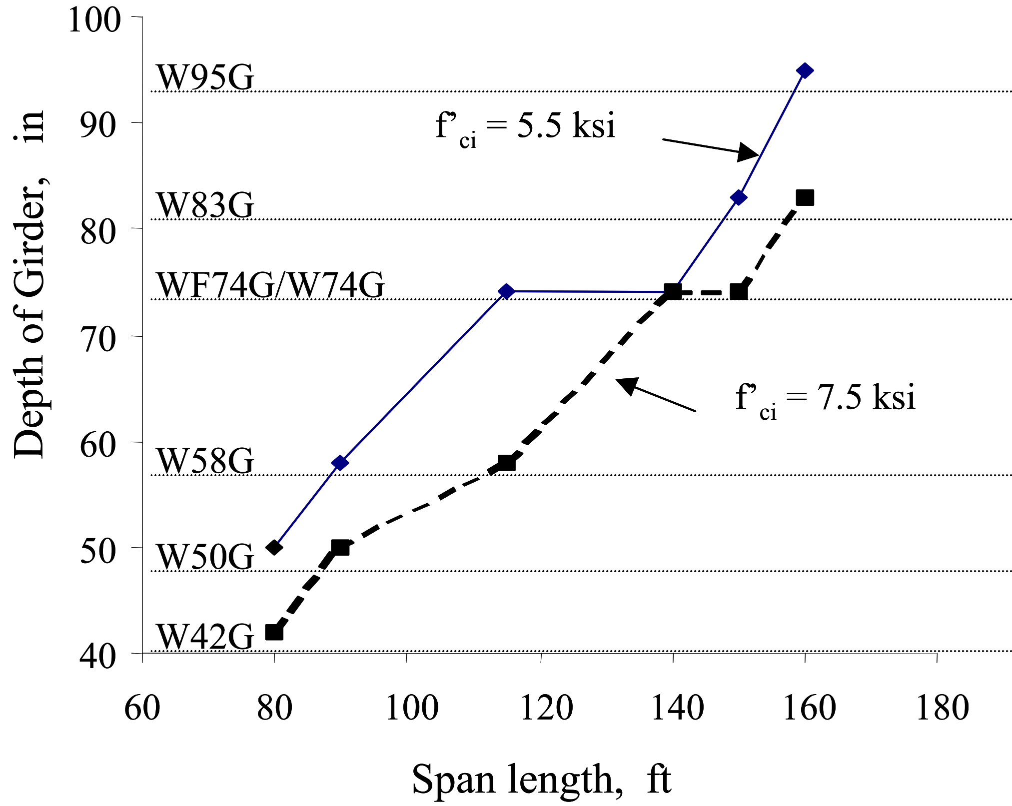

FIG. 12. Effect of Concrete Strength at Transfer on Girder Depth

1 ksi = 6.89 MPa

1 ft = 0.3048 m

1 in = 25.4 mm

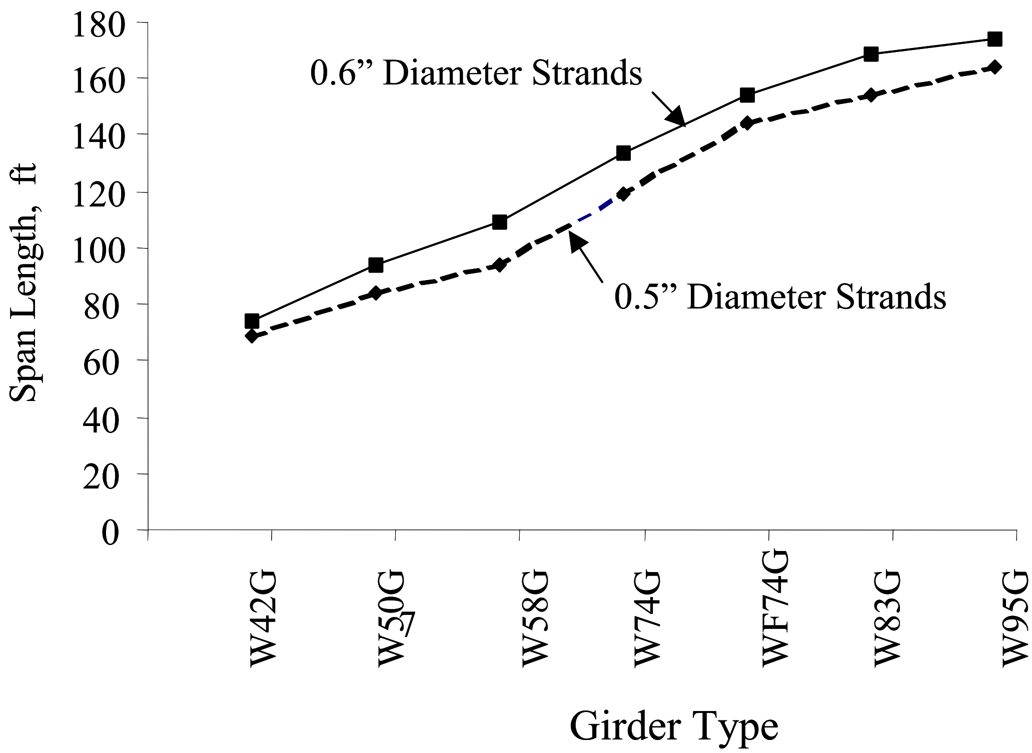

FIG. 13. Effect of Strand Diameter on Span Capability of WSDOT Pretensioned I-Girder

1 ksi = 6.89 MPa

1 FT = 0.3048 m

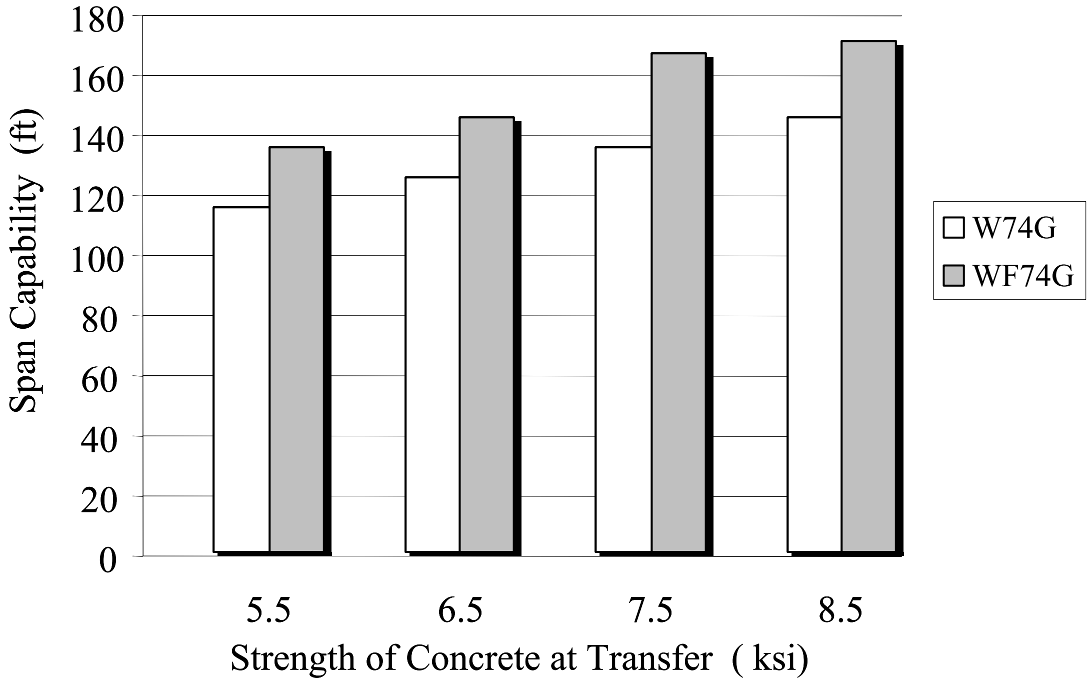

FIG. 14. Comparison of Span Capability of W74G vs. WF74G

1 ksi = 6.89 MPa

1 ft = 0.3048 m

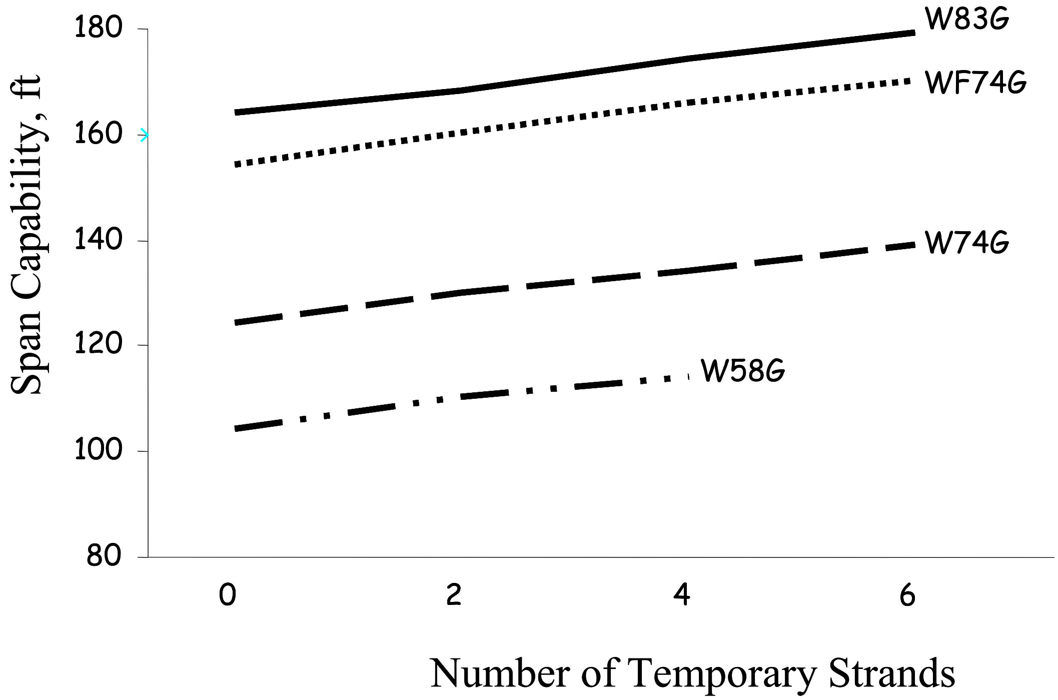

FIG. 15. Effect of Top Temporary Strands on Span Capability of WSDOT Pretensioned I-Girder

1 ft = 0.3048 m