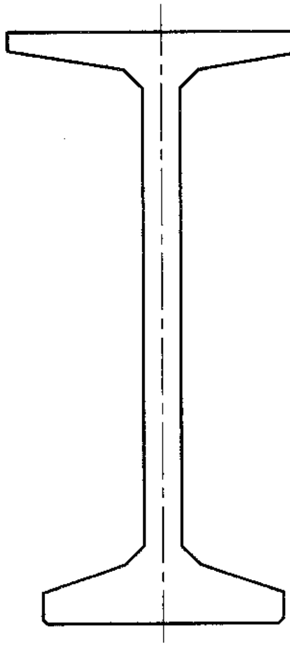
WSDOT W83MG DESIGN AIDS



CONTENTS

USE OF WSDOT W83MG DESIGN AIDS

Attention: The Design Aids included herein were derived from computer-calculated data, are intended as aids to preliminary sizing, and must be interpreted on the basis of sound engineering judgment.

INTRODUCTION

The W83MG pretensioned girder has been developed to increase the span capability of standard WSDOT prestressed concrete girders. This girder can also be used to increase the girder spacing in spans currently within the range of the W74G. The concrete outline was developed in hard SI units, and the girder was originally designated as the W21MG (2100 mm depth) to differentiate it from the smaller WSDOT standard sections, whose concrete outlines are in U.S. Customary units. However, the potential for confusion with the smaller standard sections prompted a change in the designation to include the approximate girder depth in inches. For the purposes of this brochure, SI units will be used for girder dimensions, while U.S. Customary units will be used for all other engineering information.

A W95MG pretensioned section (2400 mm depth) is also available. However, because of the increased weight, it can only be fabricated and shipped in one piece for spans of approximately 170 ft or less. The primary use of the 2400 mm deep section (W95PTMG) is for long span, post-tensioned, segmental construction. A segmental version of the 2100 mm deep section (W83PTMG) is also available for spans beyond the range of the pretensioned section.

The intent of this brochure is to provide the engineer with the information and aids needed to design a bridge superstructure with the standard W83MG section. Additional information on the development of the girder sections can be found in a paper published in the *PCI Journal*, "New Deep WSDOT Standard Sections Extend Spans of Prestressed Concrete Girders"[1]. The design assumptions used to develop the charts in this brochure differ slightly from those used in the PCI Journal article, and are based on experience gained in the design of an actual bridge superstructure.

Standard plans in both SI and U.S. Customary units have been developed for the pretensioned W83MG and W95MG girders, including standard details of intermediate and end diaphragms. Electronic copies of these standard plans are available through Concrete Technology Corporation or WSDOT's Bridge & Structures Office.

MATERIAL PROPERTIES

Concrete - The relatively high level of pretensioning will normally require the use of high strength concrete. The span capability charts were developed using $f'_c = 10.0$ ksi. For simple span designs allowing no tension in the precompressed tensile zone at service, the span capability is not particularly sensitive to concrete strength. The design concrete strength should be specified as the highest of the following: 1) the required strength at release of prestress, 2) the required strength during shipping or, 3) the required strength under service loads. The specified design concrete strength should not exceed $f'_c = 10.0$ ksi. The unit weight of concrete is assumed to be 156 pcf for calculation of the modulus of elasticity, and 160 pcf for calculating weights, including the weight of the reinforcing steel.

The concrete strength at release of prestress varies from a minimum of f'_{ci} = 4.0 ksi to a maximum of f'_{ci} = 8.5 ksi. The span capability charts show dotted lines indicating required release strengths of f'_{ci} = 7.0 ksi and f'_{ci} = 8.5 ksi. Girders with required release strengths of f'_{ci} = 7.0 ksi or less can be fabricated on a routine daily basis. Girders with required release strengths between f'_{ci} = 7.0 ksi and f'_{ci} = 8.5 ksi can be fabricated with an extended curing period. The specified concrete release strength should not exceed f'_{ci} = 8.5 ksi.

Prestressing Strand - The prestressing strand is 0.6 in. diameter, uncoated, seven-wire, low-relaxation strand meeting the requirements of *AASHTO M203* Grade 270. The initial jacking stress is $0.75f_{po}$ = 202.50 ksi. Table 1 lists the strand combinations and eccentricities used to develop the span capability charts. The variable "e" is the eccentricity of the total prestressing force from the center of gravity of the bare concrete girder between the harp points. The strand pattern between harp points is shown in Fig. 1, and the harped strand pattern at the end is shown in Fig. 2. The variable F_b is held at 3 in. for all strand configurations. The harped strands should be kept as low as possible at the ends while still maintaining the concrete stresses within allowable limits (the variable F_o varies). The slope on the harped strands should not exceed 8 horizontal to 1 vertical. The harping points are located longitudinally at 40% of the span length from the center of bearing.

Total Number of Strands	Straight Strands	Harped Strands	*E* (in.)	*F* ℄ (in.)	*e* (in.)
30	20	10	2.36	2.95	37.10
32	20	12	2.36	2.95	37.08
34	22	12	2.51	2.95	37.00
36	24	12	2.62	2.95	36.93
38	24	14	2.62	3.37	37.76
40	26	14	2.73	3.37	36.71
42	28	14	2.81	3.37	36.66
44	28	16	2.81	3.69	36.53
46	30	16	2.89	3.69	36.49
48	32	16	2.95	3.69	36.46
50	32	18	2.95	3.94	36.35
52	34	18	3.13	3.94	36.25
54	36	18	3.28	3.94	36.16
56	36	20	3.28	4.13	36.07
58	38	20	3.42	4.13	35.99
60	40	20	3.54	4.13	35.92
62	40	22	3.54	4.29	35.85
64	42	22	3.66	4.29	35.78
66	44	22	3.85	4.29	35.66
68	44	24	3.85	4.43	35.61

Table 1 - Strand Configurations and Eccentricities

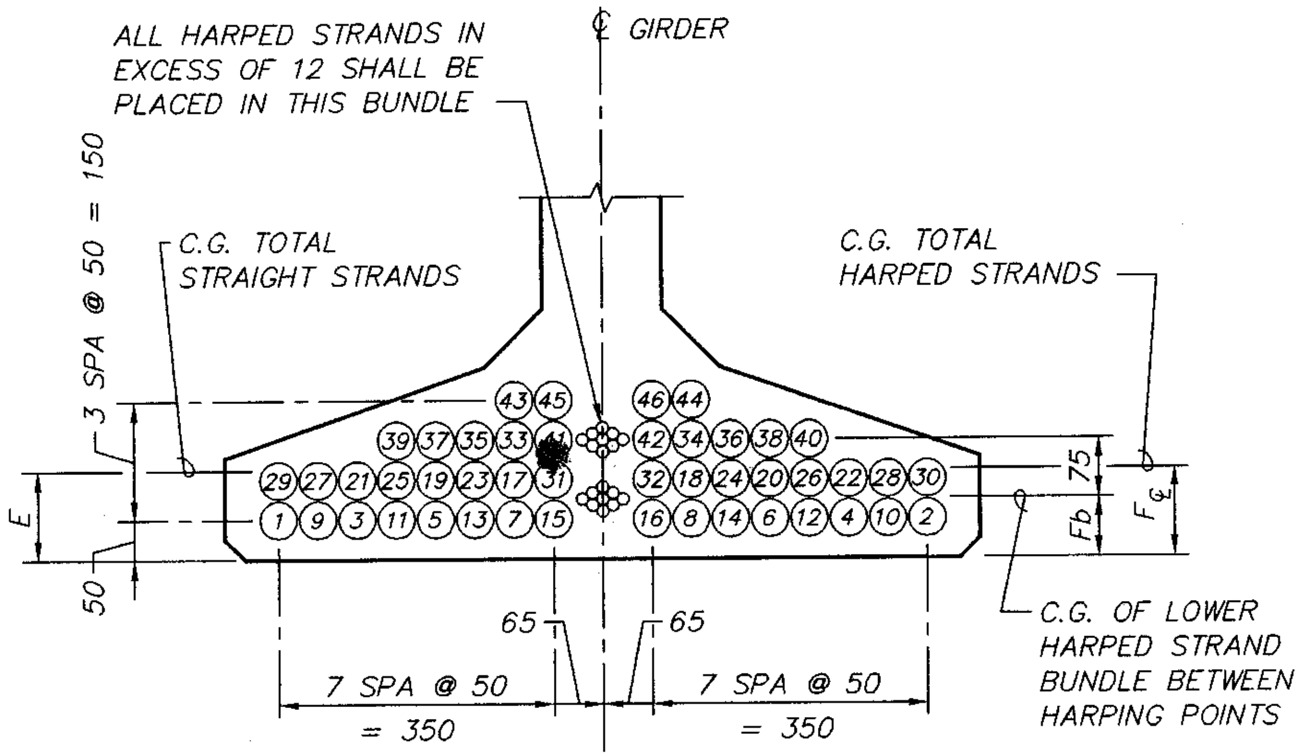


Fig. 1 — Strand Configuration Between Harp Points

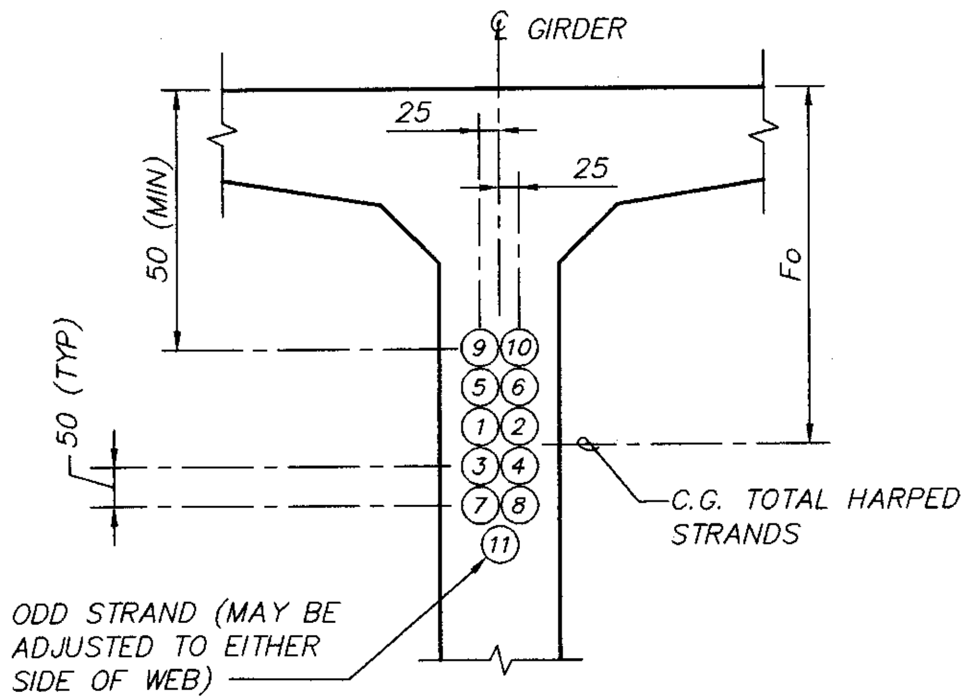


Fig. 2 — Harped Strand Configuration at Ends

DESIGN CRITERIA

The span capability charts were developed in accordance with the *1994 AASHTO LRFD Bridge Design Specifications*[2]. Comparisons[1] with designs in accordance with the *1996 AASHTO Standard Specifications for Highway Bridges, Sixteenth Edition*[3], indicate that, for HS 25-44 live load, the span capabilities differ by only a small amount. Therefore, the span capability charts in this brochure can be used to give a reasonable estimate of span capabilities using either specification. Simple spans were assumed for all loads. The maximum single piece girder weight is limited to 200 kips.

Dead Loads - Dead loads considered to act on the bare girder section included the girder, deck, 40 psf for the concrete pad between the top of the girder and the deck, and concrete diaphragms (5 ft deep by 10 in. thick) at 40 ft on center maximum.

Superimposed Dead Load - Dead load considered to act on the composite section included 40 psf for barriers.

Vehicular Live Load - *AASHTO* HL-93, including 33% impact on the truck portion only.

Limit States - Service-I (Compression) and Service-III (Tension), Strength-I. For the Strength-I limit state, it was determined[1] that *AASHTO LRFD* Article 5.7.3 underestimates the flexural strength of the composite deck-girder system. The strain compatibility method given in Section 8.2.2.5 of the *PCI Bridge Design Manual*[4] is recommended for this analysis. Consideration of compression in the top flange of the girder, and compression in the mild reinforcement in both the deck and top flange, in addition to compression in the deck, demonstrated that the deck-girder system has adequate flexural strength throughout the entire range of the span capability charts.

Live Load Distribution - *AASHTO LRFD* approximate method for both flexure and shear, interior beams.

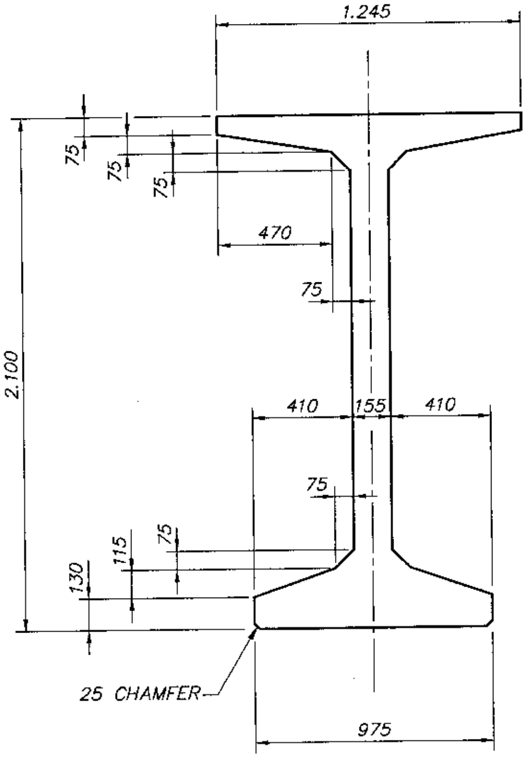
Deck Thickness - 8.86 in. (225 mm) unshored, reduced by a 0.39 in. (10 mm) wearing surface for composite section properties.

Prestress Losses - *AASHTO LRFD* Approximate Lump Sum Method or *AASHTO LRFD* Refined Method, whichever resulted in less calculated loss.

Allowable Stresses (Service) - Three charts (Figs. 3-5) are provided to reflect varying levels of allowable tension in the precompressed tensile zone; zero, $3\sqrt{f'_c}$ and $6\sqrt{f'_c}$ (for f'_c in psi). WSDOT currently allows zero tension under service loads. *AASHTO LRFD* allows $3\sqrt{f'_c}$ tension in environments with severe corrosive conditions, and $6\sqrt{f'_c}$ tension in locations with normal exposure conditions. Compressive stresses due to permanent loads are limited to $0.45(f'_c)$, while compressive stresses due to all loads are limited to $0.60(f'_c)$.

Allowable Stresses (at Release of Prestress) - Tension stresses at release are limited to $7.5\sqrt{f'_{ci}}$ (for f'_{ci} in psi) with bonded mild reinforcement to resist the total tension calculated on the basis of an uncracked section. Compressive stresses are limited to $0.60(f'_{ci})$.

GIRDER DIMENSIONS AND SECTION PROPERTIES



A	=	972 in^2
I	=	956,329 in^4
y_b	=	39.66 in
y_t	=	43.02 in
S_b	=	24,113 in^3
S_t	=	22,231 in^3
I_y	=	71,914 in^4
V/S	=	3.16 in
w	=	1.08 k/lf

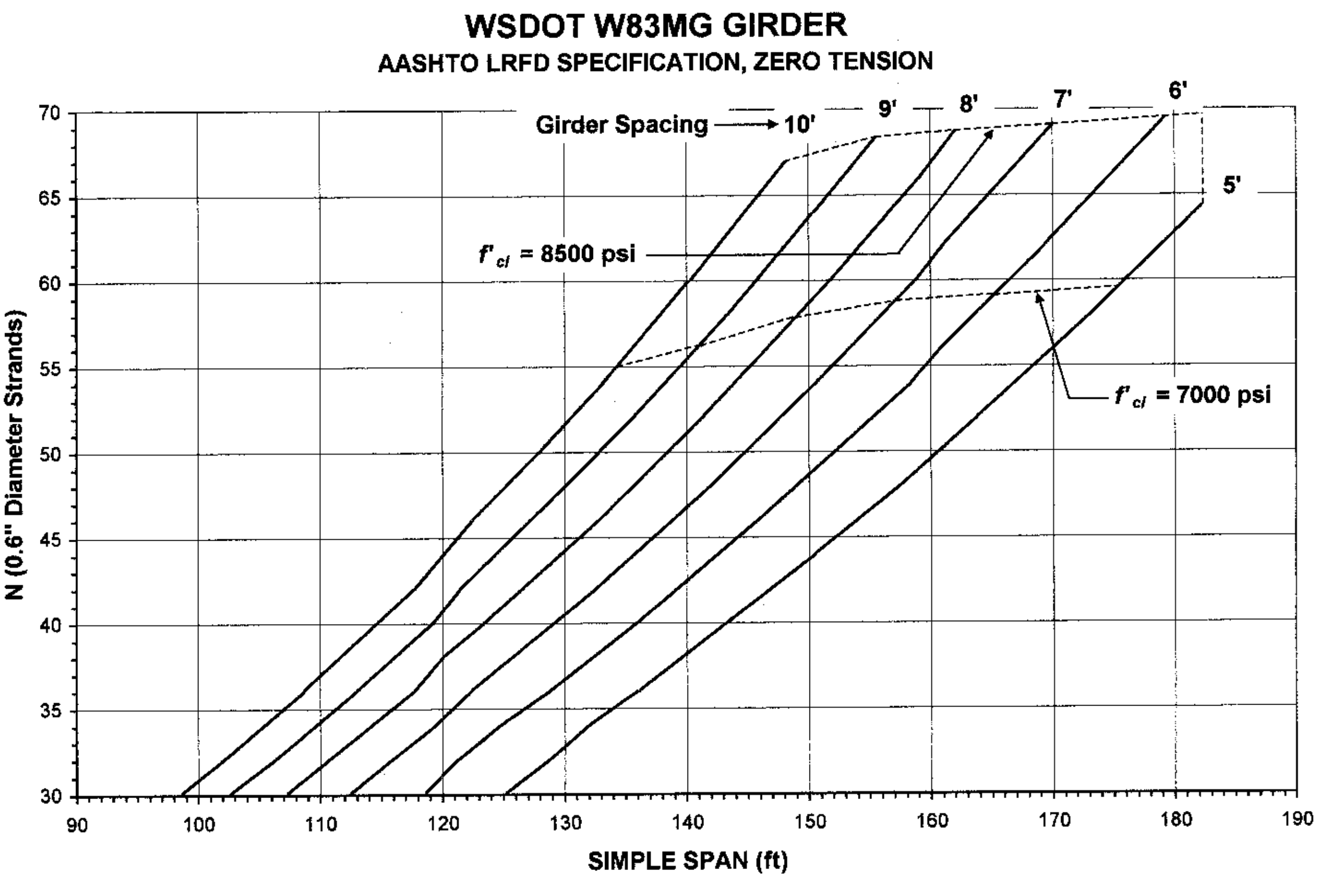


Fig. 3 — W83MG Span Capability with Zero Allowable Tension

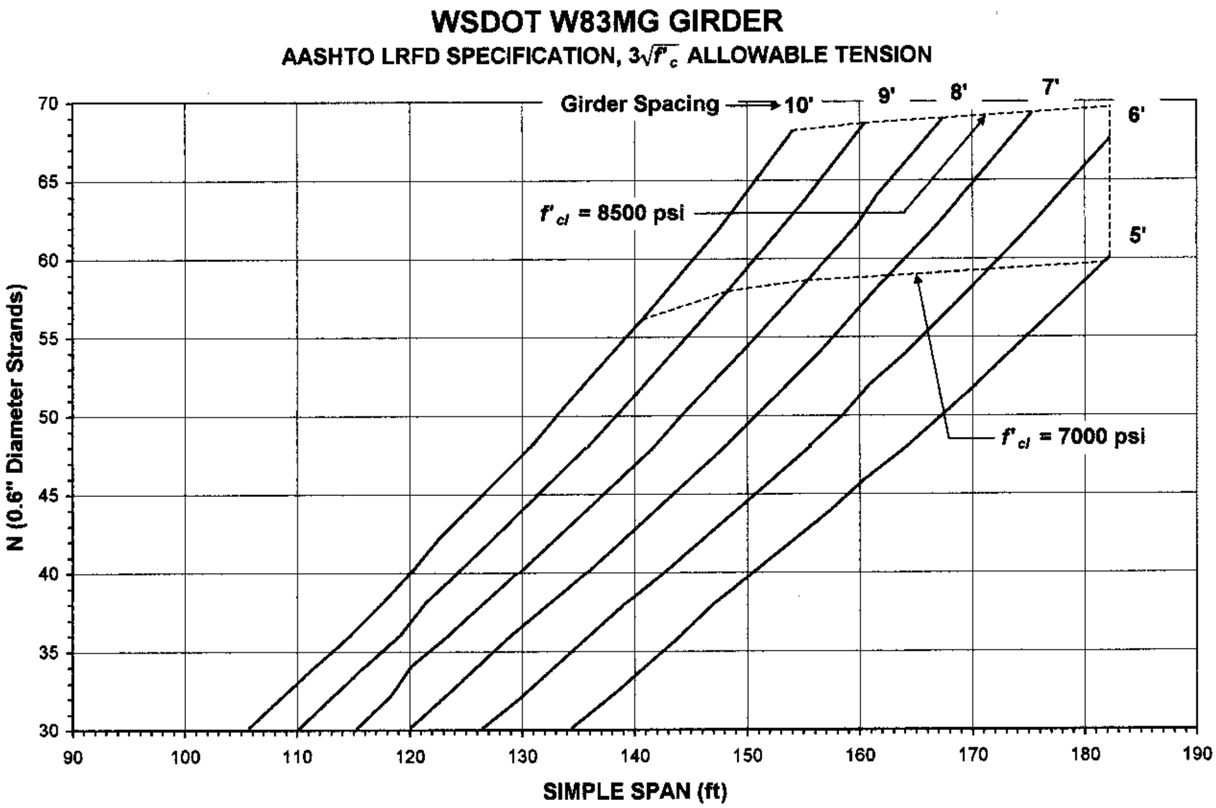


Fig. 4 — W83MG Span Capability with $3\sqrt{f'_c}$ Allowable Tension

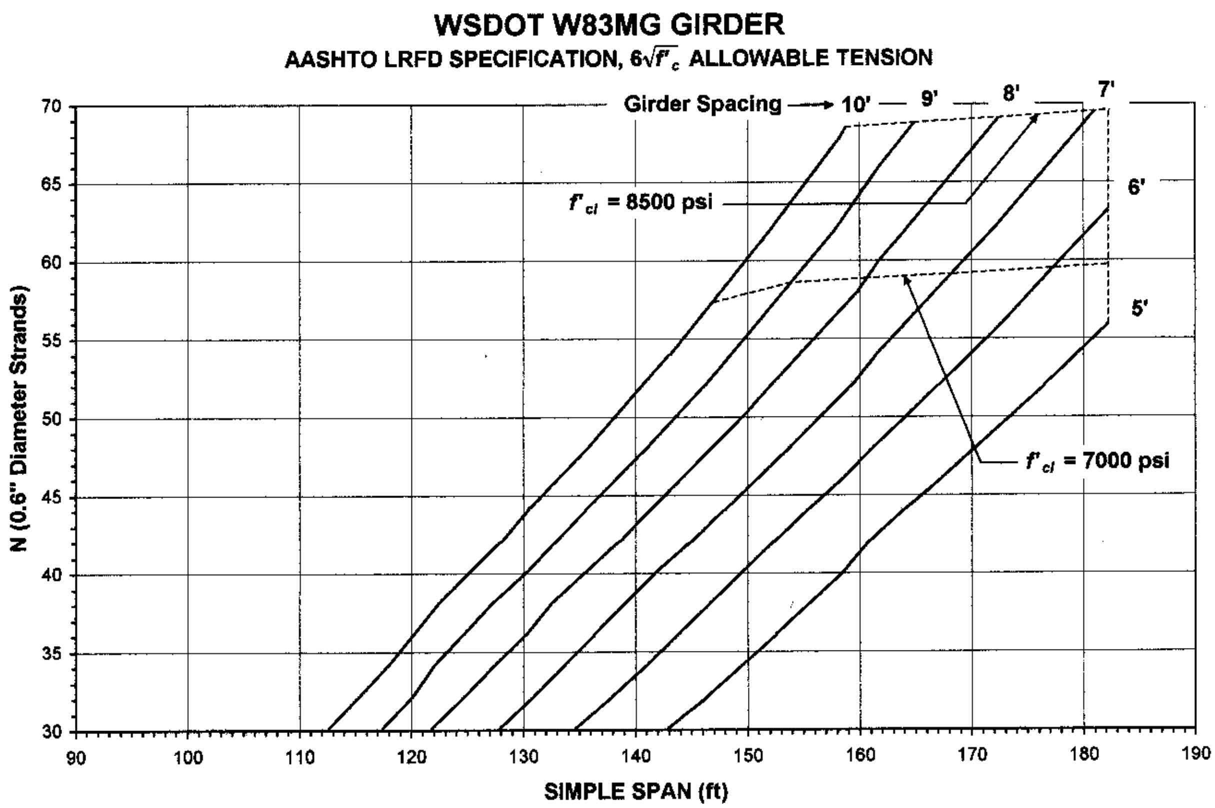


Fig. 5 — W83MG Span Capability with $6\sqrt{f'_c}$ Allowable Tension

SHEAR REINFORCEMENT

The end grid of shear reinforcement shown in Fig. 6, and in the standard plans, was developed to resist bursting and splitting forces resulting from 68 pretensioned strands (46 straight and 22 harped) plus 6 temporary strands in the top flange. This is anticipated to be the most severe condition for all practical uses of the W83MG. Beyond the end region, the demand for shear reinforcement varies significantly depending primarily on the girder spacing. Girders at narrow spacings will require only nominal shear reinforcement beyond the end region, while girders at wide spacings will require tight stirrup spacings well out into the span. For this reason, the standard plans do not call out stirrup spacings beyond the end region. The design of shear reinforcement beyond the end region must be determined by analysis for each individual design case.

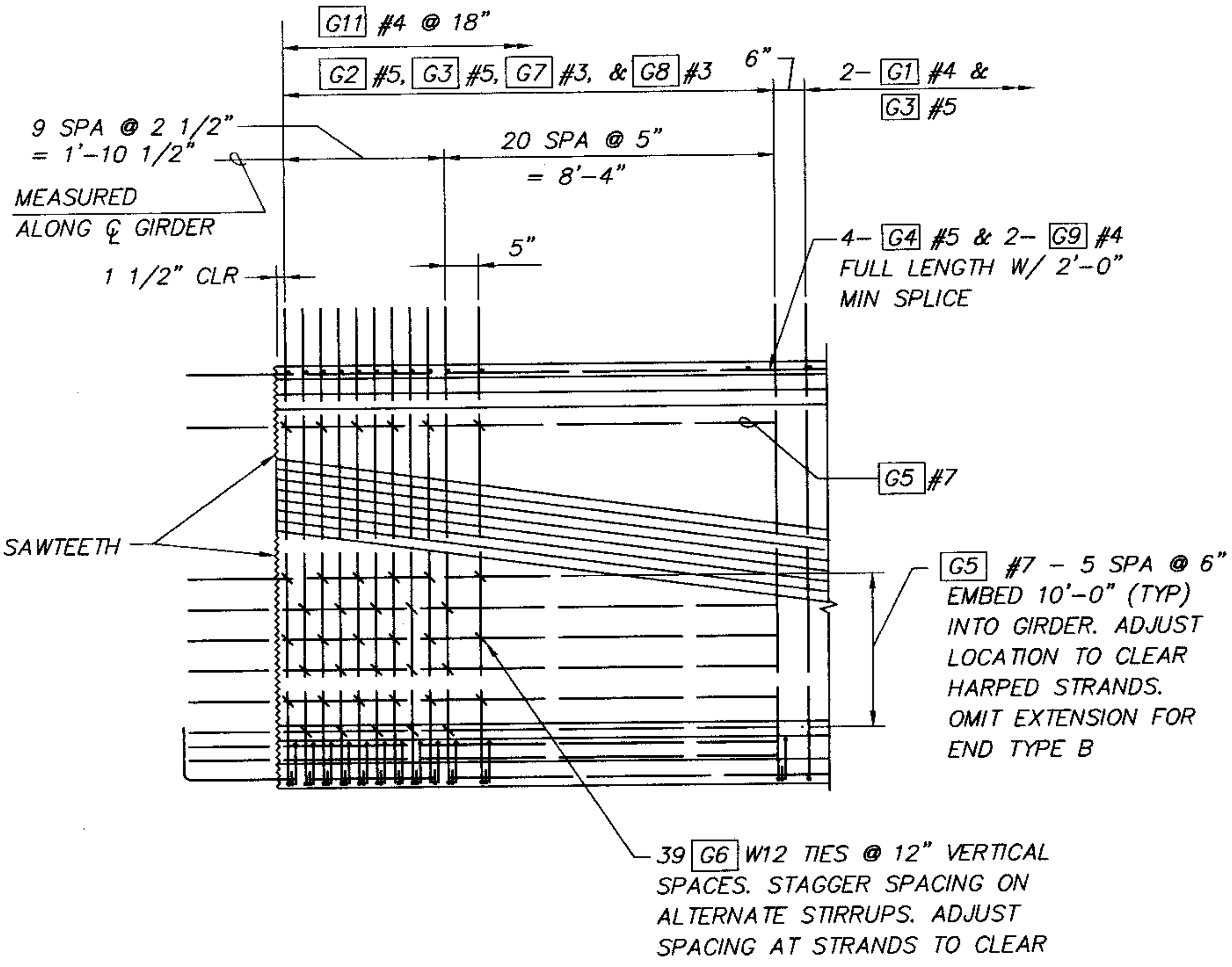


Fig. 6 — Standard Shear Reinforcement at W83MG Girder End
(See Standard Plans for Bending Diagrams)

HANDLING

The handling of these long slender girders is such an important aspect of design that aids are provided to help with the analysis. The lateral stability of a girder during lifting is influenced primarily by the location of the lifting devices with respect to the girder ends, the concrete release strength, and by bracing. CTC uses temporary strands in the top flange to improve the lateral stability factors of safety. These strands **must be cut** at the jobsite prior to placing the cast-in-place deck.

Calculation methods[1,5,6,7] have been developed to analyze the lateral stability of a long slender girder during handling. Given the girder length and number of strands, Figs. 7 and 8 can be used to plot the distance from the girder ends to the lifting devices, and the corresponding concrete release strength, that results in the recommended minimum factors of safety of 1.0 against cracking and 1.5 against failure. Fig. 7 assumes no temporary strands in the top flange, while Fig. 8 assumes 6 temporary top strands. The envelopes have been expanded to include all levels of allowable tension shown in the span capability charts. For purposes of these charts, the girder lengths are assumed to be 2.5 ft longer than the span lengths.

Fig. 7 shows that most of the girder configurations in the span capability envelope can be handled without temporary top strands, though the top portion of the envelope is truncated by the 8.5 ksi release strength. A comparison of Figs. 7 and 8 shows that temporary top strands will allow the lifting devices to be moved closer to the girder ends while maintaining the minimum lateral stability factors of safety. Compression in the bottom flange at the harp point usually governs the release strength, and increasing the span between lifting devices will increase the dead load moment and reduce the required release strength. Also, the eccentricity of the prestress is temporarily raised, which also contributes to the reduction of the required release strength. Since most girder configurations will require temporary top strands for shipping (see the next section), it makes sense to stress them prior to lifting to reduce the required release strength. Once the lifting devices have been located, and the required release strength established, the exit location of the harped strands can be adjusted downward so that the concrete stresses at the lifting locations are roughly equivalent to the stresses at the harp point. This will not affect the required release strength, and will reduce the demand on the pretensioning system.

Another advantage of the temporary top strands is the reduction of long term camber. The creep component of camber is reduced during the period between the release of prestress and pouring of the cast-in-place deck. Since the top flange is relatively wide, the concrete "pad" between the top of the girder and the deck can add a substantial amount of dead load. Consideration of the temporary top strands in design will reduce the calculated "A" dimension.

Temporary top strands can be pretensioned or post-tensioned at CTC's option. Pretensioned top strands are bonded for 10' from each end, and are unbonded for the remainder of the girder length. Post-tensioned strands are anchored only at the ends with plates and standard strand chucks. Small expanded polystyrene blockouts are provided in the top of the flange for access to cut the strands. The optimum time to cut the strands is after the cast-in-place diaphragms are cast and cured, and just prior to placing the deck.

The weight of these girders will generally preclude the use of strand lifting loops. Instead, a pair of 1- 3/8" diameter Dywidag bars at each end will most likely be required. CTC will loan the contractor lifting assemblies to be used during erection. The lifting angle must be within 10° of perpendicular to the top flange unless other arrangements are made.

W83MG GIRDER HANDLING

f'_{ci} = 4000 psi min., L = 1.75 ft min, No Temporary Top Strands

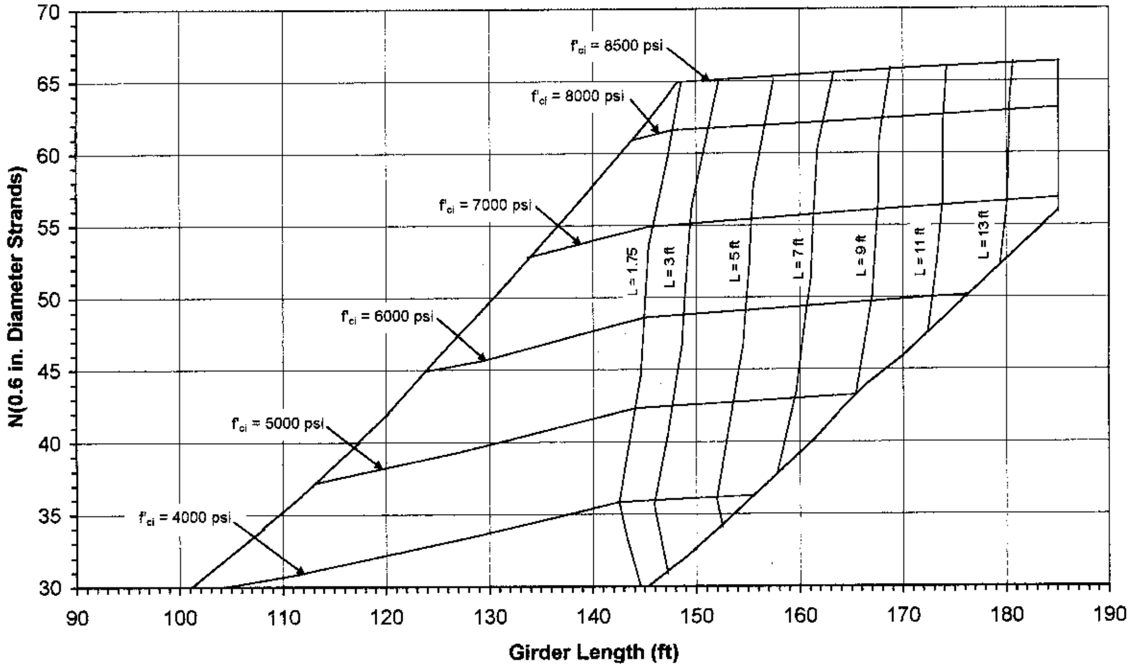


Fig. 7 — Handling with No Temporary Top Strands

W83MG GIRDER HANDLING

f'_{ci} = 4000 psi min., L = 1.75 ft min, (6) Temporary Top Strands

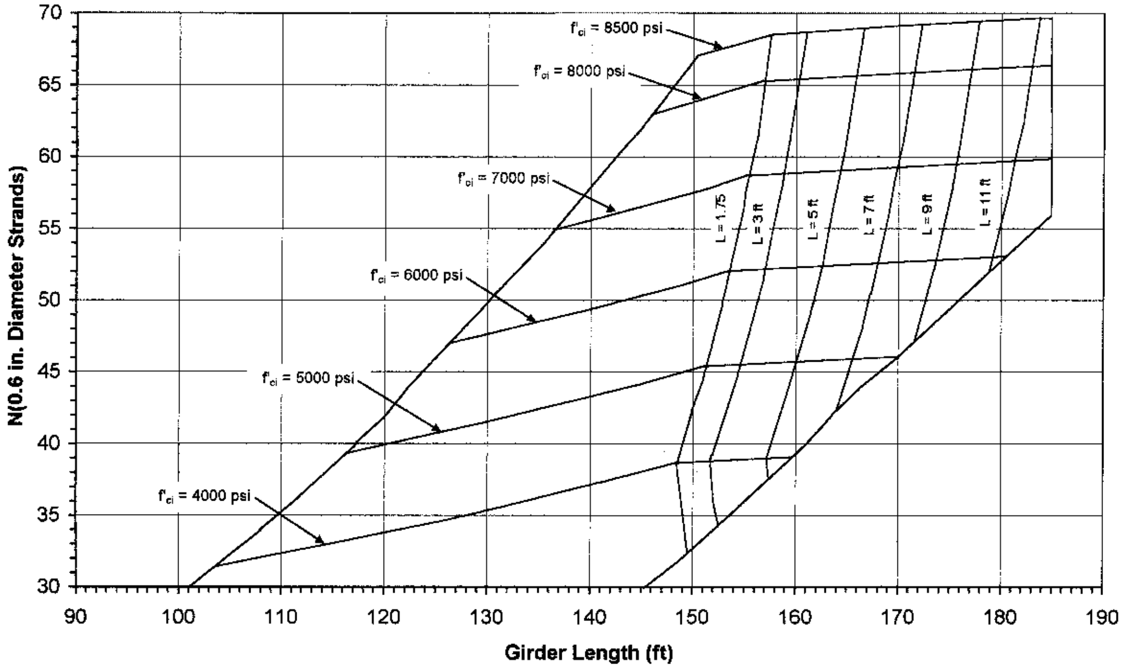


Fig. 8 — Handling with 6 Temporary Top Strands
(N does not include temporary strands)

SHIPPING

Shipping is as critical, if not more critical, than handling. The vast majority of these long slender girders must be delivered by truck. Many factors must be considered when shipping by truck; weight, length, height, and lateral stability. The ability to deliver a girder to a particular project is strongly site dependent. Routes to the site should be researched as part of the preliminary design phase.

As with handling, calculation procedures[1,6,7] have been developed for the lateral stability of girders sitting on flexible supports. Again, the factor of safety against cracking should be a minimum of 1.0, and the factor of safety against failure should be a minimum of 1.5. However, failure of a girder sitting on a truck is dominated by rollover of the truck, rather than lateral bending of the beam itself. Many girders have been shipped in Washington State with a factor of safety against cracking of less than 1.0. Factors affecting rollover of the truck include superelevation, height of the girder's center of gravity above the road, height of the truck's roll center above the road, the truck's wheel base, and the tractor-trailer rotational stiffness.

Measurements taken at CTC indicate that a truck outfitted to carry an 180,000 lb. girder had a roll center 2 ft above the road, a wheel base of 6 ft, and a rotational stiffness of approximately 41,000 in.-kips/radian. The tops of the truck supports are approximately 6 ft above the road. Local truckers like to keep the maximum distance between supports at 130 ft for turning radius purposes. It is reasonable to assume that these minimum parameters will be maintained for shipping long W83MGs.

Fig. 9 shows the required concrete strength at shipping based on the data listed above, 6 temporary top strands, and a maximum superelevation of 6%. For the entire envelope, the required concrete strength at shipping does not exceed f'_c = 10.0 ksi. Fig. 10 shows the required number of temporary top strands if a girder is shipped at f'_c = 10.0 ksi. No more than 6 temporary top strands are required for the entire envelope. It is most advantageous to use 6 temporary top strands, and to specify the required concrete strength at shipping as the design strength.

REFERENCES

1. Seguirant, S.J., "New Deep WSDOT Standard Sections Extend Spans of Prestressed Concrete Girders," PCI JOURNAL, V. 43, No. 4, July-August 1998, pp. 92-119.
2. AASHTO, *LRFD Bridge Design Specifications*, First Edition, American Association of State Highway and Transportation Officials, Washington, D.C., 1994.
3. AASHTO, *Standard Specifications for Highway Bridges*, Sixteenth Edition, American Association of State Highway and Transportation Officials, Washington, D.C., 1996.
4. *PCI Bridge Design Manual*, Precast/Prestressed Concrete Institute, Chicago, IL, 1997.
5. Mast, R.F., "Lateral Stability of Long Prestressed Concrete Beams, Part 1," PCI JOURNAL, V. 34, No. 1, January-February 1989, pp. 34-53.
6. Mast, R.F., "Lateral Stability of Long Prestressed Concrete Beams, Part 2," PCI JOURNAL, V. 38, No. 1, January-February 1993, pp. 70-88.
7. Imper, R.R., and Laszlo, G., "Handling and Shipping of Long Span Bridge Beams," PCI JOURNAL V. 32, No. 6., November-December 1987, pp. 86-101.

W83MG GIRDER SHIPPING

6% Superelevation, 130 FT Between Supports, (6) Temporary Top Strands

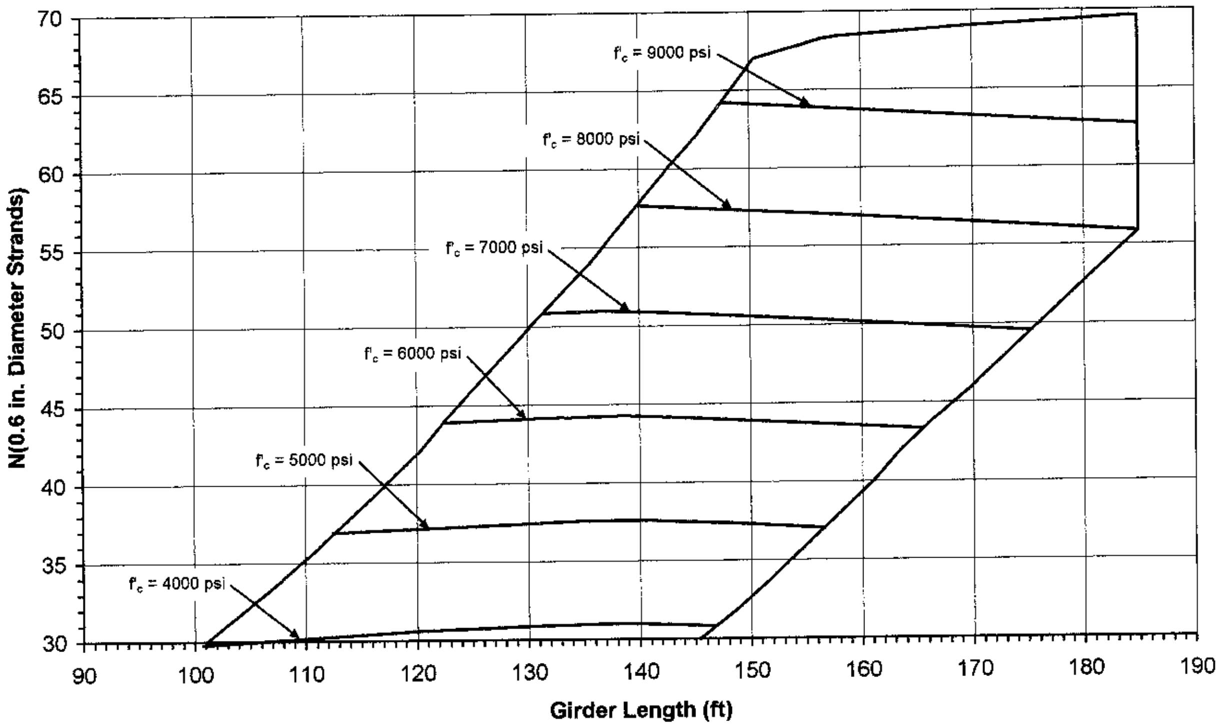


Fig. 9 — Required Concrete Strengths at Shipping with 6 Temporary Top Strands
(N does not include temporary strands)

W83MG GIRDER SHIPPING

6% Superelevation, 130 FT Between Supports, f'_c = 10,000 psi

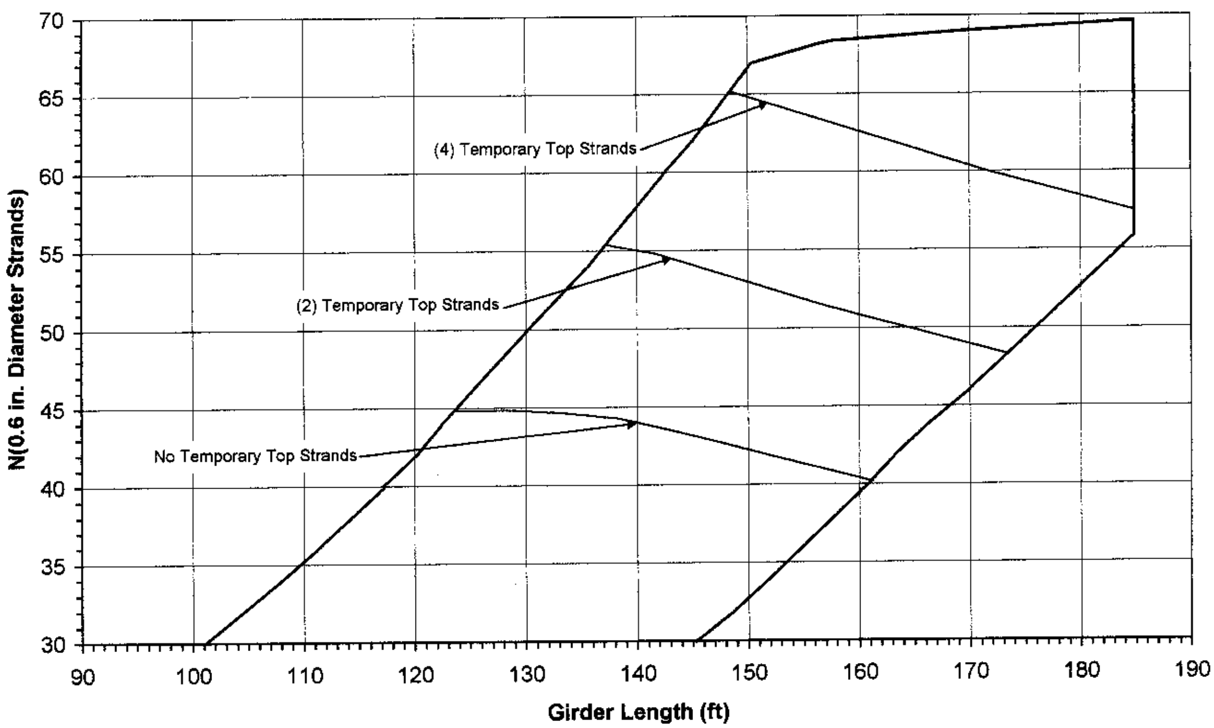


Fig. 10 — Temporary Top Strand Requirements with f'_c = 10.0 ksi at Shipping
(N does not include temporary strands)