Design Memorandum

TO: All Design Section Staff
FROM: Bijan Khaleghi
DATE: February 2, 2018
SUBJECT: Positive Moment Stand Extension

This design memorandum specifies required design criteria for determining the number of extended pretensioning strands required at integral piers with pretensioned concrete superstructures, including I and WF girders, deck bulb tee girders, tub girders, and spliced girders without post-tensioning at piers. This memorandum replaces BDM Section 5.1.3.D.3 in its entirety. BDM Appendix 5-B10 (Design Example) is deleted and will be revised in the next BDM update.

Bridge Design Manual Revisions:

BDM Section 5.1.3.D.3:

3. Strand Development Outside of Prestressed Concrete Girders

Extended bottom prestress strands are used to connect the ends of girders with diaphragms and resist loads from creep effects, shrinkage effects, and positive moments due to seismic demand.

Extended strands must be developed in the short distance within the diaphragm. Strands shall be extended as far across the diaphragm as practical, and shall be anchored at least 1’-9” from the girder end. The pattern of extended strands and embedded length of extended strands shall be sufficient to resist concrete breakout from the face of the crossbeam, while at the same time minimizing congestion. An explicit concrete breakout check may be unnecessary when all strands are effectively spliced across a crossbeam.

Strands shall be anchored with a strand chuck as shown in Figure 5.1.3-1. Strand chucks shall be a minimum 1 $^{11}/_{16}$”ø barrel anchor or similar. The designer shall calculate the number of extended straight strands needed to develop the required moment capacity at the end of each girder. The number of extended strands shall not be less than four.

For fixed intermediate piers in Seismic Design Categories B-D at the Extreme Event I limit state, the girder anchorage with extended strands shall be sufficient to carry a calculated fraction of the plastic overstrength moment demand originating from the nearest column. The required number of extended strands, N_{ps}, for each girder shall be calculated using the following:

$$N_{ps} \geq \frac{M_{u,i}}{0.9\phi A_{ps} f_{py} d} \geq 4 \qquad (5.1.3\text{-}1)$$

Where:

$M_{u,i}$ = design moment at the end of each girder. (kip-in.)

A_{ps} = area of each extended strand. (in.^{2})

f_{py} = yield strength of prestressing steel. (ksi)

ϕ = flexural resistance factor, 1.0

d = distance from the top of bridge deck to center of gravity of the extended strands. (in.)

The design moment at the end of each girder shall be calculated using the following:

$$M_{u,i} = M_{g,i} - 0.9M_{SIDL} \tag{5.1.3-2}$$

Where:

$M_{g,i}$ = The moment demand due to column plastic overstrength in girder i caused by the longitudinal seismic demands. (kip-in.)

M_{SIDL} = moment demand due to super imposed dead loads (traffic barrier, sidewalk, etc.) per girder. (k-in.)

For spliced prestressed concrete girders, where post-tension strands are applied with continuity tendons over intermediate piers, $M_{u,i}$ shall be modified to account for imposed compressive stresses.

The moment demand due to column plastic overstrength in each girder shall either be determined from the table in Appendix 5.1-A9 or Equation 5.1.3-3. This methodology assumes half the column plastic overstrength moment is resisted by the girders on each side of the column.

$$M_{g,i} = KM_{CG}\frac{\sinh\left(\frac{\lambda L_{cb}}{2N_L}\right)}{\sinh(\lambda L_{cb})}\cosh\left[\lambda L_{cb}\left(1-\frac{L_{cb,i}}{L_{cb}}\right)\right] \tag{5.1.3-3}$$

Where:

K = span moment distribution factor. If the span lengths differ, the moment contribution to each span should be modified in accordance with the span lengths, using K_1 and K_2 as shown in Figure 5.1.3-2; otherwise K = 0.5.

M_{CG} = moment generated by a single column due to the column plastic overstrength and acting at the center of gravity of the superstructure. See Equation 5.1.3-4. (kip-in.)

$L_{cb,i}$ = distance from the centerline of nearest column to centerline of the girder. (ft)

λL_{cb} = ratio of total stiffness of all girders (within a half column spacing or overhang) to torsional stiffness of half the total length of the crossbeam or half the column spacing. See Equation 5.1.3-5.

L_{cb} = half of the crossbeam length for single column bents, or half the column spacing or overhang length for multi-column bents. (ft)

N_L = the number of contributing girder lines taken as L_{cb}/S.

S = girder spacing. (ft)

The moment demand at the center of gravity of the superstructure for each column shall be calculated using the following:

$$M_{CG} = M_{po}^{top} + \frac{\left(M_{po}^{top}+M_{po}^{base}\right)}{L_c} h \tag{5.1.3-4}$$

Where:

M_{po}^{top} = plastic overstrength moment at top of column. (kip-in.)

M_{po}^{base} = plastic overstrength moment at base of column. (kip-in.)

h = distance from top of column to C.G. of superstructure. (ft)

L_c = column clear height used to determine overstrength shear associated with the overstrength moment. (ft)

The total girder stiffness to crossbeam stiffness ratio shall be calculated using the following:

$$\lambda L_{cb} = \sqrt{\left(\frac{\alpha EI}{L_g}\right)\frac{2N_L}{(GJ/L_{cb})}} \tag{5.1.3-5}$$

Where:

α = 3 for girders in which far end is free to rotate (expansion piers); and 4 for girders in which far end is fixed against rotation (continuous piers).

EI = flexural stiffness of one girder, including composite deck. (kip-in.^{2})

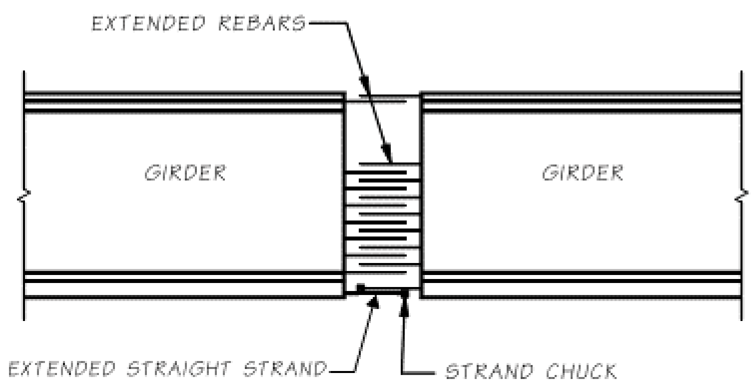
GJ = torsional stiffness of the crossbeam cross-section. (kip-in.^{2})

L_g = girder span length if girders frame into the crossbeam from only one side;

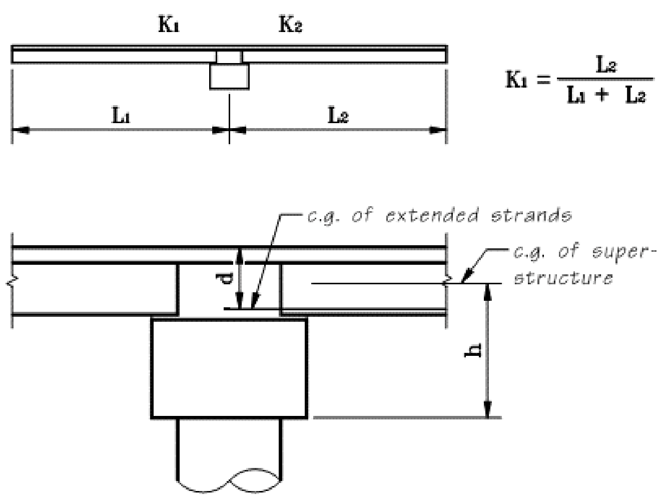
= $\frac{2}{\left({}^{1}/_{L_1}+{}^{1}/_{L_2}\right)}$, if girders frame into the crossbeam from both sides, where L_1 and L_2 are individual girder span lengths. (ft)

For dropped (two-stage) prismatic crossbeams, the moment distribution is likely to be nearly uniform. For raised (flush) crossbeams, it is likely that λL_{cb} will be > 1.0 and the moment distribution will not be uniform. For tapered crossbeams, Equation 5.1.3-2 may be used if the torsional stiffness is initially defined by the deepest section of the crossbeam, and λL_{cb} is then increased by 20%. This will lead to a less uniform distribution of girder moments than that found with a prismatic crossbeam.

A slight downwards adjustment in the number of extended strands for an individual girder is acceptable if the sum of the adjusted total moment resistance is greater than the ideal total moment resistance. Girders closer to the pier columns shall not have fewer strands than the ideal number required. When girder designs in a span are otherwise identical, the pattern and number of extended strands should also be identical, using the largest number of strands required for any girder.



Strand Development
Figure 5.1.3-1

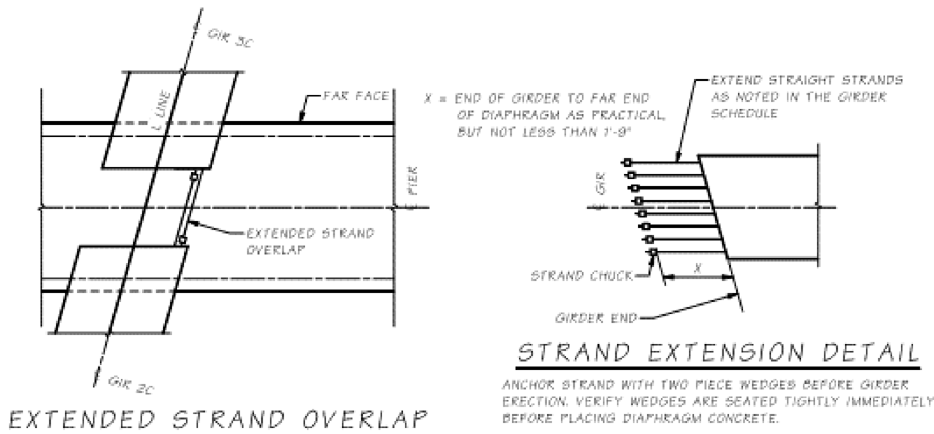


$$K_1 = \frac{L_2}{L_1 + L_2} \qquad K_2 = \frac{L_1}{L_1 + L_2}$$

Extended Strand Design
Figure 5.1.3-2

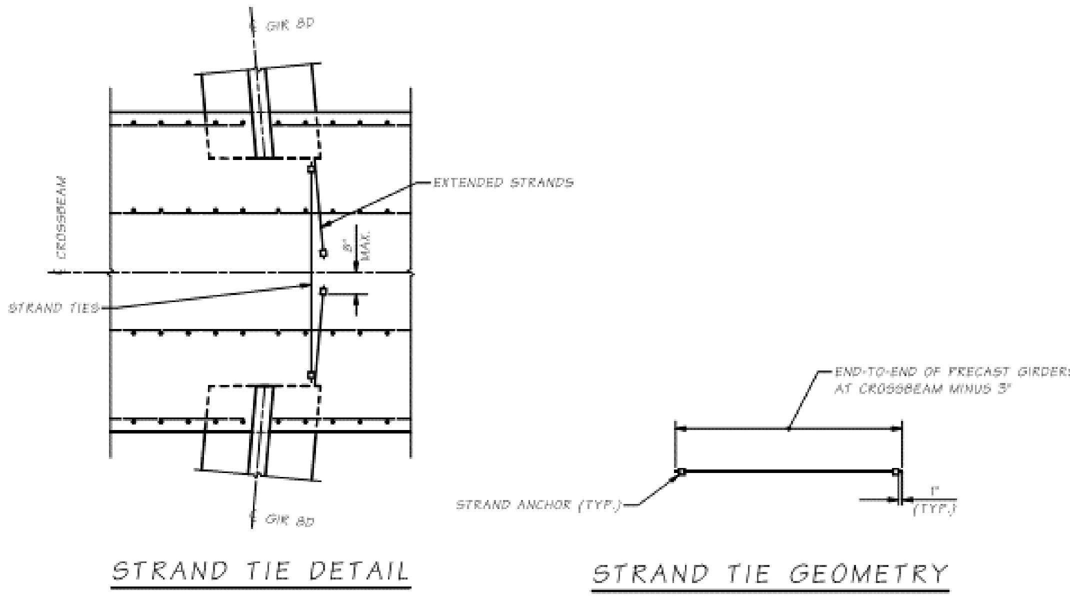
Anchorage of extended strands is essential for all prestressed concrete girder bridges with fixed diaphragms at intermediate piers. Extended strand anchorage may be achieved by providing continuity with directly overlapping extended strands, by use of strand ties, by the use of the crossbeam ties along with strand ties, or by a combination of all three methods. The following methods in order of hierarchy shall be used for all prestressed concrete girders for creating continuity of extended strands:

Method 1 - Direct extended strands overlapping shall be used at intermediate piers without any angle point due to horizontal curvature and for any crossbeam width. This is the preferred method of achieving extended strand continuity. Congestion of reinforcement and girder setting constructability shall be considered when large numbers of extended strands are required. In these cases, strand ties may be used in conjunction with extended strands. See Figure 5.1.3-3.



Overlapping Extended Strands
Figure 5.1.3-3

Method 2 - Strand ties shall be used at intermediate piers with a girder angle point due to horizontal curvature where extended strands are not parallel and would cross during girder placement. Crossbeam widths shall be greater than or equal to 6 feet measured along the skew. It is preferable that strand ties be used for all extended strands, however if the region becomes too congested for rebar placement and concrete consolidation, additional forces may be carried by crossbeam ties up to a maximum limit as specified in Equation 5.1.3-6. See Figure 5.1.3-4.



Strand Ties
Figure 5.1.3-4

Method 3 - For crossbeams with widths less than 6′ and a girder angle point due to horizontal curvature, strand ties shall be used if a minimum of 8″ of lap can be provided between the extended strand and strand tie. In this case, the strand ties shall be considered fully effective. For cases where less than 8″ of lap is provided, the effectiveness of the strand tie shall be reduced proportional to the reduction in lap. All additional forces not taken by strand ties must be carried by crossbeam ties up to the maximum limit as specified in Equation 5.1.3-6. If this limit is exceeded, the geometry of the width of the crossbeam shall be increased to provide sufficient lap for the strand ties. See Figure 5.1.3-5.

The area of transverse ties, A_s, considered effective for strand ties development in the lower crossbeam shall not exceed:

$$A_s = \frac{1}{2}\left(\frac{A_{ps} f_{py} N_{ps}}{f_{ye}}\right) \qquad (\text{in.}^2) \qquad (5.1.3\text{-}6)$$

Where:

A_{ps} = area of strand ties. (in.^{2})

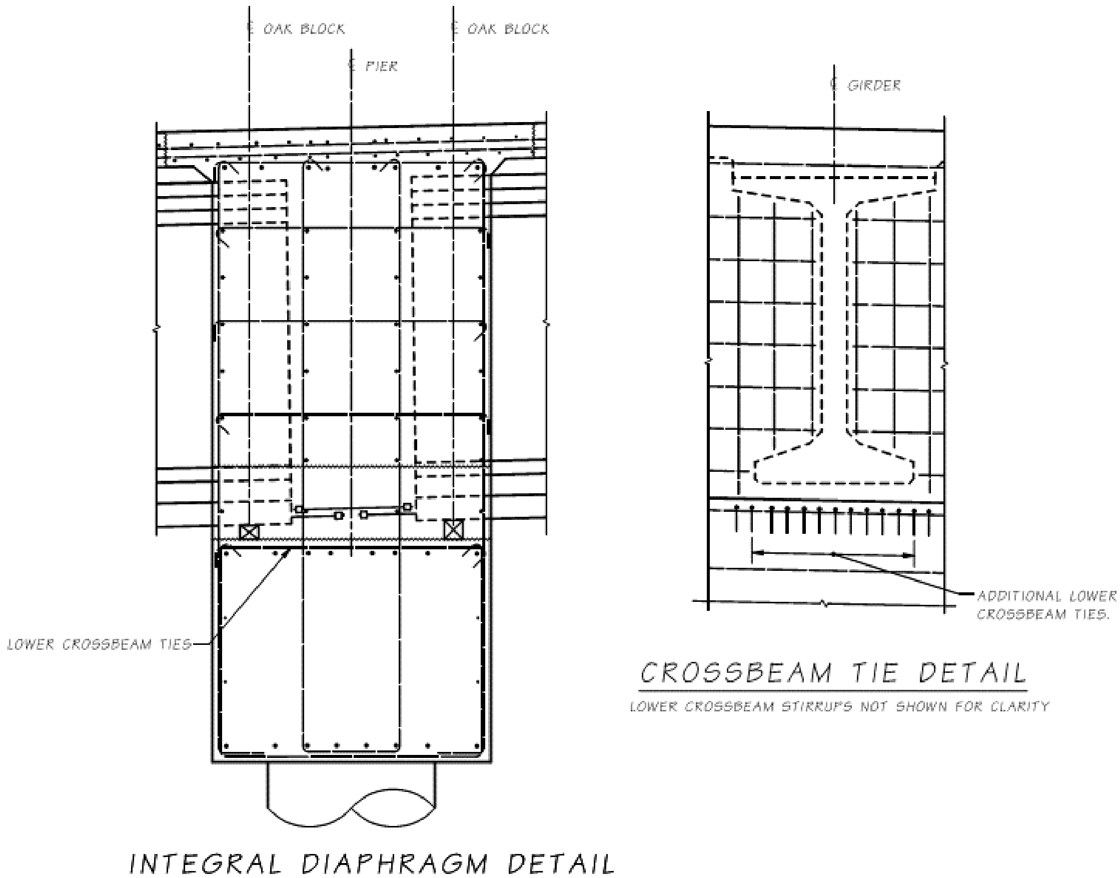
f_{py} = yield strength of extended strands. (ksi)

N_{ps} = number of extended strands that are spliced with strand and crossbeam ties.

f_{ye} = expected yield strength of transverse tie reinforcement. (ksi)

Two-thirds of A_s shall be placed directly below the girder and the remainder of A_s shall be placed outside the bottom flange width as shown in Figure 5.1.3-5.

The size of strand ties shall be the same as the extended strands, and shall be placed at the same level and proximity of the extended strands.



Lower Crossbeam Ties
Figure 5.1.3-5

BDM Appendix 5.1-A9 is added:

Girder Moment Ratios as a Function of the Stiffness Ratio, λL_{cb}

λL_{cb}	$N_L = 2$		$N_L = 3$			$N_L = 4$			
	$\frac{M_{g,1}}{\frac{1}{2}M_{CG}}$	$\frac{M_{g,2}}{\frac{1}{2}M_{CG}}$	$\frac{M_{g,1}}{\frac{1}{2}M_{CG}}$	$\frac{M_{g,2}}{\frac{1}{2}M_{CG}}$	$\frac{M_{g,3}}{\frac{1}{2}M_{CG}}$	$\frac{M_{g,1}}{\frac{1}{2}M_{CG}}$	$\frac{M_{g,2}}{\frac{1}{2}M_{CG}}$	$\frac{M_{g,3}}{\frac{1}{2}M_{CG}}$	$\frac{M_{g,4}}{\frac{1}{2}M_{CG}}$
0	0.250	0.250	0.167	0.167	0.167	0.125	0.125	0.125	0.125
0.2	0.251	0.249	0.168	0.166	0.166	0.126	0.125	0.125	0.124
0.4	0.255	0.245	0.172	0.166	0.163	0.129	0.126	0.123	0.122
0.6	0.261	0.239	0.177	0.164	0.158	0.135	0.126	0.121	0.118
0.8	0.269	0.231	0.185	0.163	0.152	0.142	0.127	0.118	0.113
1	0.278	0.222	0.195	0.161	0.144	0.150	0.128	0.114	0.107
1.2	0.289	0.211	0.206	0.158	0.136	0.160	0.129	0.110	0.101
1.4	0.301	0.199	0.218	0.155	0.127	0.171	0.130	0.105	0.094
1.6	0.313	0.187	0.230	0.152	0.118	0.182	0.131	0.100	0.086
1.8	0.326	0.174	0.243	0.148	0.108	0.194	0.131	0.095	0.079
2	0.338	0.162	0.257	0.144	0.099	0.206	0.132	0.090	0.072
2.2	0.350	0.150	0.270	0.140	0.090	0.219	0.131	0.085	0.065
2.4	0.362	0.138	0.283	0.136	0.081	0.231	0.131	0.080	0.058
2.6	0.373	0.127	0.295	0.132	0.073	0.243	0.130	0.075	0.052
2.8	0.384	0.116	0.307	0.127	0.066	0.255	0.129	0.070	0.046
3	0.394	0.106	0.319	0.122	0.059	0.266	0.128	0.065	0.041
3.2	0.403	0.097	0.330	0.118	0.052	0.277	0.126	0.061	0.036
3.4	0.412	0.088	0.341	0.113	0.047	0.287	0.124	0.056	0.032
3.6	0.420	0.080	0.351	0.108	0.041	0.297	0.122	0.052	0.028
3.8	0.427	0.073	0.360	0.104	0.037	0.307	0.120	0.049	0.025
4	0.434	0.066	0.369	0.099	0.032	0.316	0.117	0.045	0.022

Notes:

1. Table values represent a ratio of the girder moment demand, M_{gi}, to half the column plastic overstrength moment, $\frac{1}{2}M_{CG}$.
2. The $M_{g,i}$ subscripts 1, 2, 3 and 4 refer to the girder lines. Line 1 is located closest to the column.
3. This table is based on the following assumptions:
   a. Equal span lengths (K = 0.5)
   b. First girder located at half the girder spacing from the centerline of the nearest column.
   c. N_L is an integer (i.e. 2, 3, 4). This represent an even number of girders between columns in a multi column pier.

Background:

Precast prestressed bridges in Washington State are designed to resist longitudinal seismic forces by frame action. Moments must therefore be transferred from the ends of the girders, where they are manifested as bending, into the crossbeam, where they are manifested as torsion. Consequently, positive moment connections are needed at the girder ends, and they are presently achieved by extending some of the strands located in the bottom flange of the girder into the cast-in-place diaphragm, where they are anchored with strand vices and bearing plates. That hardware configuration can cause conflicts and difficulties during construction.

The design criteria provided are the result of a research project conducted by the University of Washington (WA-RD 867.1). The goals of the research were to investigate the existing method of making the positive moment connection and, if possible, to develop a new one that would be easier to construct. The goals were addressed using a combination of laboratory testing and structural analysis. The first two tasks, associated with the anchorage capacity of strands equipped with barrel anchors, were completed using laboratory tests. The last task, in which the moment demand at each girder end was evaluated, was conducted using structural analysis.

The performance of a single anchorage device, and in particular the possibility of local concrete crushing behind the strand anchor, was first investigated by physical testing. It was found that a barrel anchor, with no backing plate, was easily sufficient to resist the force imposed by a strand at incipient fracture.

Second, the possibility of a group of strands breaking out from the diaphragm was evaluated. Tests were carried out on strands, anchored by strand vices, grouped in various patterns and embedded to various depths in concrete blocks representing a diaphragm. It was found that the results were closely predicted by the Concrete Capacity Design method, which can therefore be used to predict the minimum embedment length required to ensure that strand fracture takes place prior to a group breakout failure mode.

Third, the distribution of girder end moments across the bridge was evaluated using Finite Element Analysis. The important parameters were the flexural properties of the girders and the torsional properties of the crossbeam. The outcome focused on the girder end moments when the column reached its full flexural capacity.

The strand anchor plate has been eliminated as recommended by the University of Washington research report. Only strand chucks (barrel anchors) as used in the specimen testing are required to develop strands for positive moment connections. Strand chucks shall be extended as far across the diaphragm as practical, but not less than 1’-9” from the girder end.

Concrete breakout resistance shall be determined by concrete capacity design methods, such as ACI 318-14 Chapter 17.

Research Conclusions

- **Resistance of a single anchor.** A barrel anchor, with no backing plate, is easily sufficient to resist, without damage to the anchor or bearing failure of the concrete, the force exerted by a strand at incipient fracture. The high resistance to bearing stresses (measured at approximately 100 ksi on concrete with f'c ≈ 4 ksi) is attributed to very efficient confinement behind the anchor.
- **Resistance of a group of strands to failure by breakout.** The Concrete Capacity Design method predicted very accurately the failure loads measured in the tests conducted for this project. Those tests, with the tests originally used to develop the method, provide verification of the method, which can therefore be used to evaluate the minimum embedment depth needed to ensure that ductile strand yield and fracture precede brittle group breakout failure.
- **Distribution of girder moment demand across the bridge.** For bridges constructed with a two-stage cap-beam, such as is used extensively in Washington State, the crossbeam system is sufficiently stiff in torsion compared to the flexural stiffness of the girders that the moment demand in the end of each girder is almost uniform across the width of the bridge. The end moment used for design, and consequently the number of strands that need to be extended from each girder, may thus be reduced below the values required by the existing design method. That method was developed from tests conducted on a system in which the ratio of crossbeam torsional stiffness to girder bending stiffness was much lower than that typical of Washington State bridges.

Research Recommendations for Implementation

- Extended strands, up to and including 0.6" diameter, should be equipped with a barrel anchor alone. No backing plate is necessary.
- Strands should be extended as far as possible across the crossbeam to achieve the best transfer of moment into the crossbeam. Terminating the strand just inside the vertical reinforcement at the far side of the crossbeam maximizes the structural benefits and minimizes the construction difficulties.
- Girder end moments may be determined using the approach outlined in Section 6.6 of this report. In most cases, that will lead to equal moments in each girder and the smallest possible number of strands extending from each girder.
- Consideration should be given to eliminating the non-prestressed bar reinforcement in the girder ends, and generating the force required for vertical shear friction in the web from the extended strands alone. This would simplify fabrication of the girders and reduce the possibility of bar conflicts on site.

Research Recommendations for Further Research

- The research conducted here addressed only right bridges. The anchorage capacities are expected to remain unchanged in skew bridges, so the primary need is to determine the moment demands in skew bridges. Preliminary hand calculations suggest that, in a skew bridge, the crossbeam behaves as if it were stiffer than in a comparable right bridge, in which case the girder end moments would be distributed even more uniformly than was found here. However, that tentative finding should be investigated using FEA.

References:

Tsvetanova, K., Stanton, J. F., Eberhard, M. O. (2017) *Developing Extended Strands in Girder-Cap Beam Connections for Positive Moment Resistance*, Report No. WA-RD 867.1, Washington State Transportation Center (TRAC)

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