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CONNECTIONS FOR RESISTING LONGITUDINAL SEISMIC LOADS IN BRIDGES MADE WITH PRETENSIONED CONCRETE GIRDERS.

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Abstract

Longitudinal seismic loads in bridges may be resisted either by the columns acting as cantilevers or by frame action between the superstructure and the columns. The latter arrangement is preferable because it reduces the size and cost of the foundations, but it requires a moment connection between the ends of the girders and the column, at each bent. For superstructures made from precast- pretensioned concrete girders, this requires connection of both the positive and negative moment steel.

Development of the top steel is relatively straightforward, using bar steel embedded in the cast-in-place deck slab. For the bottom steel (and positive moment connection), different approaches have been tried in the past, using devices such as bent-up strand, supplementary bar steel in the web, or strand anchors with backer plates seated on the extended strands. All of these have disadvantages, particularly with respect to constructability.

The study reported here focuses on two aspects of the problem. First, it was shown experimentally that a strand vice alone, with no need for a bearing plate to spread the load, provides easily enough anchorage to ensure that failure occurs by strand fracture rather than bond loss or local concrete crushing. This detail significantly reduces congesting of the hardware. Second, analytical studies addressed the present requirements in the AASHTO Seismic Guide Specifications (AASHTO, 2011) that address the number of girders that can be considered active in resisting seismic moment. The analyses showed that the AASHTO requirements are too restrictive and not rationally based. A new formulation was developed, based on the relative stiffnesses of the various components. It allows a larger number of girders to participate, and therefore a smaller number of strands in each girder to be extended into the diaphragm. That, in turn, reduces congestion and further improves constructability.

Keywords: Bridge girders; seismic; strand; development; strand vice; pier cap; torsion.

1. Introduction

Longitudinal seismic loads in bridges may be resisted either by the columns acting as cantilevers or by frame action between the superstructure and the columns. The latter arrangement is preferable because it reduces the size and cost of the foundations, but it requires a moment connection between the ends of the girders and the column, at each bent. For superstructures made from precast- pretensioned concrete girders, this requires connection of both the positive and negative moment steel.

Development of the top steel is relatively straightforward, using bar steel embedded in the cast-in-place deck slab. For the bottom steel (and positive moment connection), different approaches have been tried in the past, using devices such as bent-up strand, supplementary bar steel in the web, or strand anchors with backer plates seated on the extended strands. All of these have disadvantages, particularly with respect to constructability.

The first goal of the study to reported here was to investigate ways of anchoring the strands that are more effective structurally and more readily constructible. The second goal was to investigate the transfer of moments from the ends of the girders into the cap beam. That process is presently controlled by requirements in the AASHTO Seismic Guide Specifications (AASHTO, 2011), which are quite restrictive in terms of the effective width of deck within which the girders must lie in order to be considered as contributing to the longitudinal seismic resistance. Those restrictions mean that only a small number of girders may be counted, in which case the moment carried by each one, and the number of bottom strands to be extended, become quite large. That in turn creates additional congestion and exacerbates the problem of strand anchorage.

2. Strand Anchorage

2.1. Background

Positive moment strength at the end of a girder requires tension capacity at the bottom of the girders. This is most easily provided by extending some or all of the strands in the bottom flange. The number of strands available for the purpose depends on the regional practice. In some states, the strands are divided into two groups, one consisting of fully bonded straight strands in the flanges, and the other consisting of depressed, or "harped" strands in the web. At the end of the girder, the latter are relatively high in the cross-section, so are not useful for positive moments. In other states, all of the strands are in the bottom flange region, but some are locally debonded at the girder end to avoid excessive service stress there. Both the bonded and debonded strands could, in theory, be used for strength, but in practice the bonded ones offer the better response because they will develop full stress under a smaller girder end rotation.

Present practice is to anchor the extended strands using either by a straight length or bending them up. The former arrangement is rare, because the cast-in-place diaphragm joining the opposing girders is usually too narrow to permit development of the strand within its width. In states with low seismic exposure, the development of longitudinal resistance is of low priority, so the diaphragm can be narrow (0.5 to 1.5 ft, 150 – 450 mm), in which case the strands have to bent upwards immediately after exiting the girder, as shown in Fig 1, in order to maintain a feasible bend radius. The value of the anchorage is then suspect, because the embedded length is small.

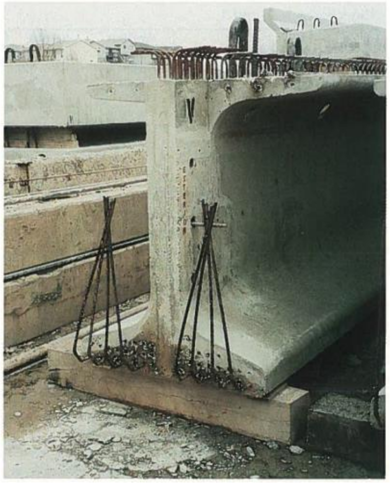

Fig. 1. Bent Extended Strand Detail.

The third option is to us straight strands equipped with a strand vice (also referred to as a "strand chuck" or "strand anchor") with a plate behind it to spread the force so that it remains within the bearing capacity of the concrete. This places a significant amount of weight at the end of the extended strand, which causes the strand to bounce significantly during struck transportation. The plate dimensions are also approximately 4"x4" (100mm x 100mm) so they prevent adjacent strands in the prevailing 2" grid from being extended and used for anchorage. The plates also cause considerable congestion during placement of the girder, fixing of the diaphragm steel and concrete casting. This is particularly true if the bridge is skewed.

The proposed research included a test program in which strands, equipped with vices with different sized plates, would be loaded in tension until the system failed. The concrete surrounding the anchor was confined with a steel tube, whose wall thickness was determined so that the lateral confinement stress in the specimen would be approximately the same as that available in the cap beam.

The test plan was to start with no plate, in order to provide a base-line, and subsequently to use increasingly large plates until the critical size was found. However, in the first test, the strand broke, with little apparent sign of damage to the concrete. After repeating that test for confirmation, further tests were conducted by loading the strand vice directly in compression from the "back", rather than by loading the strand in tension from the front. By that means, it would be possible to impose higher loads and determine the true failure load of the concrete in bearing, and thus the safety margin available if a strand vice were to be used with no plate.

2.2 Test Program

The configuration for the compression tests is shown in Fig 2. The details of the load train were changed slightly as testing progressed, largely to avoid damage to the steel components. The latter consisted of a 1.75" dia. Williams rod (f_u = 150 ksi) and one or more ASTM A 490 nuts ($f_u \geq$ 150 ksi), which were introduced in order to create a central void in the loading system where it was in contact with the strand vice, and thereby to load the body, rather than the wedges of the anchor. Details can be found in Tsvetanova et al. (2017).

The concrete cylinders in which the anchors were embedded were contained in steel tubes, specifically HSS 7x0.5 circular tubes, with ID = 6", OD = 7", and f_y = 42 ksi. This arrangement was chosen to provide a confinement stress up to 7 ksi when the tube wall yielded, which was deemed to be approximately the level of confinement to be expected in the cap beam in the field. One of the changes adopted during the progress of the tests was to cast the concrete cylinders inside plastic molds with OD slightly smaller than the ID of the steel cylinder, then to grease the plastic and insert it in the steel cylinder. The purpose was to minimize the vertical stress in the steel, so that the concrete confinement stresses could be established with greater confidence from the readings of strain gages on the outer face of the steel.

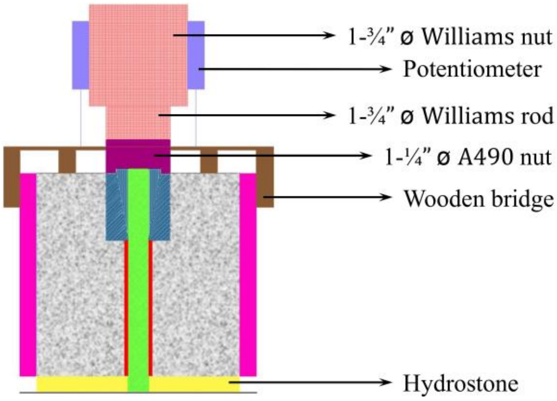

Figure 2. Setup for Compression Tests.

2.3 Test Results

The test configurations and results are shown in Table 1 and Table 2. The concrete was cast in two batches, which had 28-day strengths of 4700 and 5400 psi respectively for batches A and B. The anchors consisted of barrel anchors (circular with outside diameter of 1.6875" and a central opening of 0.625", net bearing area = 1.930 in^2) and "casting anchors", which are typically used for unbonded post-tensioning of slabs, are made from cast steel and have an integral flange measuring 3"x5.875". The bearing stresses under the casting anchors were much lower than those under the barrel anchors, so only the latter are considered here. The acronyms for the specimens are:

TBA = **B**arrel **A**nchor loaded in **T**ension
CBA = **B**arrel **A**nchor loaded in **C**ompression
CCA = **C**asting **A**nchor loaded in **C**ompression

The peak loads for the barrel anchors loaded in tension (not shown) were approximately 60 kips, and were controlled by strand fracture. The peak loads for the barrel anchors loaded in compression (CBA1 – CBA6) lay between 194 and 249 kips. However, these values are lower bounds to the concrete failure stress, because in all cases loading was stopped because the load train failed by yielding and deforming, despite being made of very high strength steel. In one case, the strand anchor was cut out of the test specimen after testing was complete, and the only noticeable concrete damage was a layer, approximately 0.08% (2mm) thick, beneath the bearing interface, in which the cement paste had been crushed to a powdery consistency. Other than that, no concrete damage was visible.

Even though the capacity of the concrete was never reached, the peak loads were still 4.0 to 4.5 times higher than the load required to fracture the strand in tension. It is therefore evident that the effects of confinement are sufficient to provide a bearing capacity that is substantially higher than necessary, and that there is no danger of the concrete suffering a bearing failure if a barrel anchor is used with no backing plate.

Table 1. Specimen Properties

Anchor name	f'c	load train	load locn.	tube length	greased/ dry
(-)	(ksi)	(-)	(-)	(in)	(-)
TBA 1	4.7	strand	N/A	6	dry
TBA 2	4.7	strand	N/A	6	dry
CBA 1	4.7	rod	wedge	6	dry
CBA 2	4.7	rod, nut	barrel	6	dry
CBA 3	4.7	rod, nut, pl	barrel	6	dry
CBA 4	5.4	rod, 4 nuts	barrel	6	greased
CBA 5	5.4	rod, 4 nuts	barrel	6	greased
CBA 6	5.4	rod, 4 nuts	barrel	12	greased

Table 2. Test Results.

Test	P_{max} (kips)	f'_c (psi)	$f'_{bearing}$ (ksi)	ε_{vert} (με)	ε_{hoop} (με)	σ_{hoop} (ksi)	f'_L (ksi)	$\frac{f'_{bearing} - f'_c}{f'_L}$
CBA[1] 1	249.3	5400	129.2	-	578.1	18.42	3.070	40.31
CBA 2	205.6		106.6	-	536.4	17.09	2.849	35.51
CBA 3	220.5		114.3	-	748.9	23.86	3.977	27.37
CBA 4	193.5	4700	100.3	-158.3	577.0	16.87	2.812	33.98
CBA 5	200.4		103.9	-103.3	591.4	17.86	2.977	33.32
CBA 6	225.3		116.8	-165.3	873.4	26.25	4.375	25.61
CCA[2] 1	199.7	5500	11.62	-	249.9	7.962	1.327	4.613
CCA 2	204.7		11.91	-	290.8	9.267	1.544	4.151
CCA 3	212.7		12.38	-	324.0	10.32	1.721	3.997

[1]CBA = Barrel Anchor in Compression
[2]CCA = Casting Anchor in Compression

In order to investigate the interaction between the lateral confining stress and the bearing stress, the confining stress was calculated from the hoop strain in the steel, and the confinement coefficient, c_L, was calculated as shown in Table 2, where

$$f'_{cc} = f'_c + c_L f'_L \tag{1}$$

and

f'_c = 28-day unconfined strength
f'_{cc} = 28-day confined strength
f'_L = lateral confinement stress.

For c_L, the commonly accepted value (ACI 318-14) is 4.1, based on the work by Richart, Branzaeg and Brown (1929). The values calculated from the barrel anchor tests lay in the range 27 – 40. The difference is attributed to the fact that the load under the barrel anchors is applied only over an area that is much less than the total area, in which case the effects of confinement are greatly amplified. ACI 318-14 recognizes this and allows an increase in the bearing stress, but to a much smaller extent than was found here. And it should be recalled that the true c_L values would be higher still, if the loading system had not failed before the specimen. It should also be noted that the hoop stress in the steel tube never exceeded about half of the yield stress.

The observed behavior, and the measured test values, showed that the bearing capacity of the barrel anchors was easily sufficient to sustain the necessary loads dictated by strand fracture. The behavior also showed that bearing stresses much higher than those predicted by conventional criteria are possible under special circumstances of confinement, such as those used here, and that the matter merits further investigation.

3. Contribution of Girders to Longitudinal Resistance.

3.1 Background.

Resistance to longitudinal seismic loads comes from shear in the columns. That shear induces column moments. While it is possible to design the column to work as a cantilever, practice in some states, such as California, is to minimize the moments at the base in order to save on foundation costs. The connection between the column and cap beam must then resist all the moment. That connection must also resist moment if the column is designed to resist bending at both top and bottom. The bending moment at the top of the column is transferred in torsion along the cap beam to the girders, where it is once again manifested as a bending moment. Torsional flexibility in the cap beam will cause the girders closer to the column to resist a greater proportion of the column moment.

The AASHTO Seismic Guide Specification (AASHTO, 2011) contains rules for determining an effective width, B_{eff}, centered on the column. The girders falling within that width are treated as resisting 2/3 of the total column moment, with the remaining one sixth resisted by the rest of the girders. However, WSDOT requires that all girders have the same number of extended strands, which is controlled by the requirement on the innermost girders. In most cases, only two girder lines fall within B_{eff}, so each of the girder lines must have enough extended strands to resist one third of the column moment. (Note that, in any one girder line, two girders meet, end-to-end, at the cap beam, so each girder must carry one sixth of the total moment.) That condition then controls the number of extended strands in all girders.

The effective width is defined as

$$B_{eff} = D_c + D_s \tag{2}$$

where D_c and D_s are the depth of the column and superstructure respectively. The concept is illustrated in Fig (3).

Those rules were derived from experimental work by Holombo et al (2000), on two test structures typical of California practice of the day. The superstructure of one consisted of two box girders, placed symmetrically with respect to the column, and the other consisted of six I-girders. The definition of the effective width was based on measured strains in the longitudinal girder reinforcement.

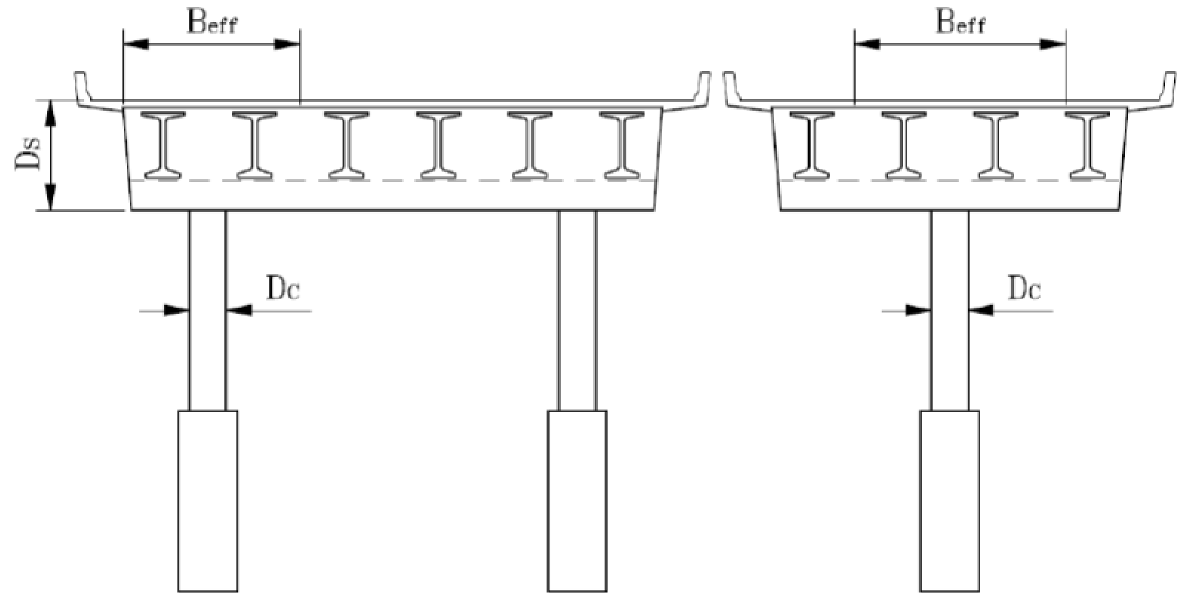

Figure 3. Definition of Effective Width.

It can be shown that, in an elastic model, the critical parameter that controls the distribution of girder moments is the ratio of the torsional stiffness of the cap beam to the flexural stiffness of the girders, expressed as

$$\lambda = \sqrt{\frac{K_g/s}{GJ}} \qquad (3)$$

where

K_g = rotational stiffness of the opposing two girders in a line that meet over the cap beam,
s = spacing of girder lines,
GJ = torsional stiffness of the cap beam.

λ has the units of length^{-1}. λL_{cb} is the relevant dimensionless constant, where L_{cb} is the length of the cap beam projecting from the column (i.e. half the total cap beam length in a single-column bent.)

California typically uses integral cap beams, which do not project below the girders, while Washington uses dropped cap beams. The latter are built in two stages. The first stage, or lower cap beam, is cast on the columns, and serves to support the precast longitudinal girders when they are placed. The second-stage cap beam, or diaphragm, is then cast on top of the first, and is made composite with it. It encases the girder ends. The Washington cap beam is therefore almost twice as deep as the California one, and has a significantly higher torsional stiffness, a lower value of λL_{cb}, and better ability to distribute the column moment over multiple girders. This reasoning led to the belief that the AASHTO design rules may be unduly restrictive for Washington bridges.

3.2 Analyses Conducted

While D_s in Eq. (2) is related to the torsional stiffness of the cap beam, it alone does not fully reflect the behavior. Consequently, analyses were conducted to investigate the true behavior of the system and to develop guidelines for design.

Three models were used:

- a finite element model using shell and solid element in the program ABAQUS,
- a "stick" model using line elements,
- a continuum model, for which a closed-form solution was developed.

Each of the two finite element models consisted of two spans supported by a central column bent. The far ends of the six girder lines were treated as pin-supported, both vertically and horizontally, and the single column rested on a roller support at the bottom and longitudinal load was applied there. The continuous model represented only the cap beam and girder connections to it, represented by torsional springs, and the loading consisted of a torsional moment applied at the column location. It may be thought of as a torsional "beam on elastic foundation" model.

The outcome depends on the values of several parameters.

- Bridge overall geometry,
- Member sizes,
- Degree of cracking,
- Extent of strand yielding at the girder ends
- Mesh size in the model,
- Use of rigid end offsets for members,
- etc.

3.3 Results of Analysis.

The analyses were calibrated against each other, and good agreement was found, especially for relatively stiff cap beams. For cap beams that are torsionally very flexible, parameters such as the number of girder lines and the width of the rigid regions ("rigid end offsets" in the models), which are expressed differently in the three models, mean that exact agreement should not be expected. The dimensionless parameter λL_{cb} was used as the independent variable, and results are plotted against that in Fig. 4.

Fig. 4 shows the ratio $M_{g,max}/M_{g,ave}$ as a function of λL_{cb}. $M_{g,max}/M_{g,ave}$ is a measure of the uniformity of girder moments across the bridge width. If λL_{cb} is small, the cap beam is torsionally stiff compared with the girder's bending stiffness, and the moment in each girder is almost the same. Then the maximum and average girder moments are nearly the same. As λL_{cb} increases, the cap beam becomes relatively more flexible in torsion, and the distribution of girder moments becomes more non-uniform. The value $\lambda L_{cb} = 0.435$ is taken from the two-span bridge with a total of six girder lines used as a prototype used for the study. It was built using WSDOT W74G girders spaced at 6.5 ft centers, with individual spans of 134 ft. Its dimensions are typical of highway overcrossings in Washington State.

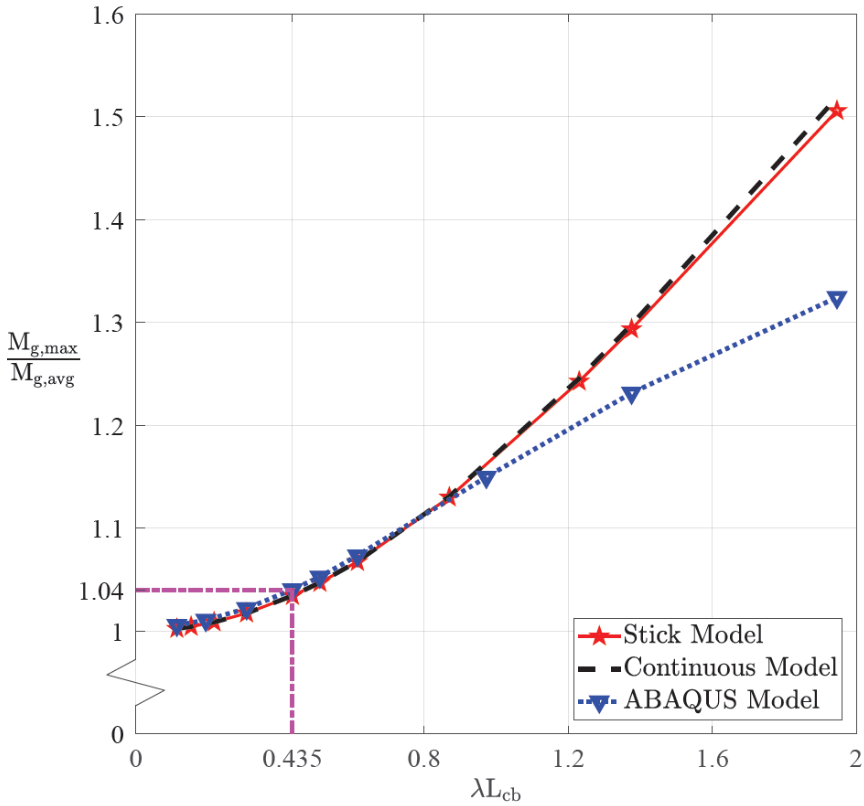

Figure 4. Maximum vs. Average Girder Moment for Different Cap Beam Stiffnesses.

In Fig. 4, the curve for the continuous model is defined by

$$\frac{M_{g,max}}{M_{g,ave}} = \frac{\lambda L_{cb}}{tanh(\lambda L_{cb})}$$

The ratio $M_{g,max}/M_{g,ave}$ is asymptotic to 1.0 when λL_{cb} approaches zero, and to λL_{cb} for large $\lambda L_{cb.}$. As can be seen, agreement with the stick model is very good throughout the range, and agreement with the ABAQUS model is also good in the practical range of $\lambda L_{cb} < 1.0$ For the prototype bridge, with λL_{cb}, the girder moments are nearly the same across the entire bridge width. This outcome differs significantly from that predicted by Holombo et al. (2000), largely because the cap beam that they tested was much smaller than those typically use in Washington State. The implication is that each girder line can be treated as each carrying 1/6 of the total column load, and each girder line anyway contains two opposing girders. This finding reduces the number of strands that must be extended from each girder, and therefore relieves the problems of anchor congestion.

4. Summary and Conclusions

4.1 Summary

A study was carried out to investigate improved ways of developing positive moment resistance at the ends of prestressed concrete girders in bridges for the purpose of improving design for longitudinal seismic load. It consisted of three components. First, tests were conducted to determine whether strand barrel anchors alone (i.e. without backing plates) could be used to anchor a strand embedded in the cast-in-place second state cap beam. Second, analyses were conducted to determine the maximum end moment in the girders caused by longitudinal seismic loading on the bridge.

4.2 Conclusions

The following conclusions were drawn.

- A conventional barrel strand anchor with no backing plate can be used to anchor the strand without risk of concrete failure in bearing. Although the calculated bearing stress on the concrete directly behind the anchor is much higher than the cylinder strength of the concrete, the effects of confinement increase the concrete strength locally so that it is easily sufficient for the need.
- For a system in which the load is applied only over part of the area, as is the case here, the confinement is much more effective than the provisions of ACI 318 suggest. That behavior merits further investigation.
- The AASHTO Seismic Guide Specifications specify an effective width of bridge deck for the purpose of determining resistance to longitudinal seismic load. Two thirds of the seismic moment from the column must be resisted by the girders lying within that effective width. This study found that that requirement is much too stringent for the bridges built in the State of Washington, which typically are constructed using a two-stage cap beam that has a high torsional stiffness. A different and more representative way of determining the contribution of each girder was developed and presented.
- The larger moments predicted by the new design guidelines to be carried by girders more distant from the column allow more girders to contribute to resisting the total column moment, thereby reducing the moment to be carried by each. That is turn reduces the number of strands that must be extended from each girder, and improves the constructability of the system.
- The recommendations from this research have been adopted by the WSDOT.

Acknowledgments.

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References

AASHTO (2011). "AASHTO Guide Specifications for LRFD Seismic Bridge Design", 2nd ed. American Association of State Highway and Transportation Officials, Washington DC.

ACI 318 (2014). " Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary." American Concrete Institute, Farmington Hill, MI.

Holombo, J., Priestley, M. J. N., and Seible, F. (2000). "Continuity of Precast Prestressed Spliced-Girder Bridges under Seismic Loads." *PCI Journal*, 45(2), 40–63.

Richart, F. E., Brandtzaeg, A., and Brown, R. L. (1929). "The Failure of Plain and Spirally Reinforced Concrete in Compression." *University of Illinois Bulletin*, Vol. 26, Engineering Experiment Station, University of Illinois at Urbana-Champaign, 1–74.

Tsvetanova, K, Stanton, J.F. and Eberhard, M.O. (2017). "Developing Extended Strands in Girder-Cap Beam Connections for Positive Moment Resistance." Research Report no. 867.1, Washington State Department of Transportation. Nov, 164p.