STRENGTH AND SERVICEABILITY OF REINFORCED CONCRETE INVERTED-T STRADDLE BENT CAPS

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ABSTRACT

Inverted-T straddle bent caps are often used in bridge construction to reduce the elevation of bridges and/or to improve available clearance beneath the beams. The bridge-deck stringers are supported on ledges at the bottom of the Inverted-T bent cap, effectively loading the caps at their tension chord. This arrangement generates a tension field in the web near loading points, as forces are 'hung' to the compression chord at the top of the beam. In contrast, top- or compression-chord loading does not generate such a tension fields in the web. Several recently built Inverted-T caps in Texas have shown significant inclined cracking triggering concern about current design procedures for such structures.

A study aimed at evaluating the differences between tension- and compression-chord loaded members is presented. Results are included from an ongoing study on Inverted-T straddle bent cap specimens (TxDOT Project 0-6416). Test specimens presented in this paper were full scale with overall height of 42 inches. Parameters varied in the tests were ledge depth, and number of loading points.

Results from a previous study on top- or compression-chord loaded specimens with similar dimensions and reinforcing details (TxDOT Project 0-5253) are compared with results from the bottom- or tension-chord loaded specimens of the current study. Preliminary findings pertaining to the strength and serviceability implications of web tension fields are discussed.

Keywords: Research, Inverted-T, Deep Beams, Shear, Straddle Bent Caps, Strut-and-Tie Modeling.

INTRODUCTION

There are 13 documented cases of Inverted-T straddle bent caps in Texas exhibiting shear cracking at early ages. The present study aims to obtain a better understanding of the structural behavior of bottom-chord loaded specimens and to develop new design criteria to minimize/eliminate such cracking in the future.

In top-chord loaded specimens loads are applied on top of the web where the force is transferred from the point of application directly to the support, via a compression strut. Alternatively, in bottom-chord loaded specimens like Inverted-T beams the loads are applied indirectly to the bottom of the web through the ledges. The forces are then "hung" up to the compression chord via stirrups and then transferred to the supports. The resulting tension field is shown in red in Figure 1.

A recent study (TxDOT Project 0-5253, *Strength and Serviceability Design of Reinforced Concrete Deep Beams*) developed design guidelines using Strut-and-Tie modeling for strength and serviceability of deep beams. This study focused on top-chord loaded members. The design provisions proposed by TxDOT Project 0-5253 were used to estimate ultimate strengths of Inverted-T specimens of the current project. A preliminary assessment of the applicability of 0-5253 provisions to tension-chord loaded members was performed to identify any required modifications.

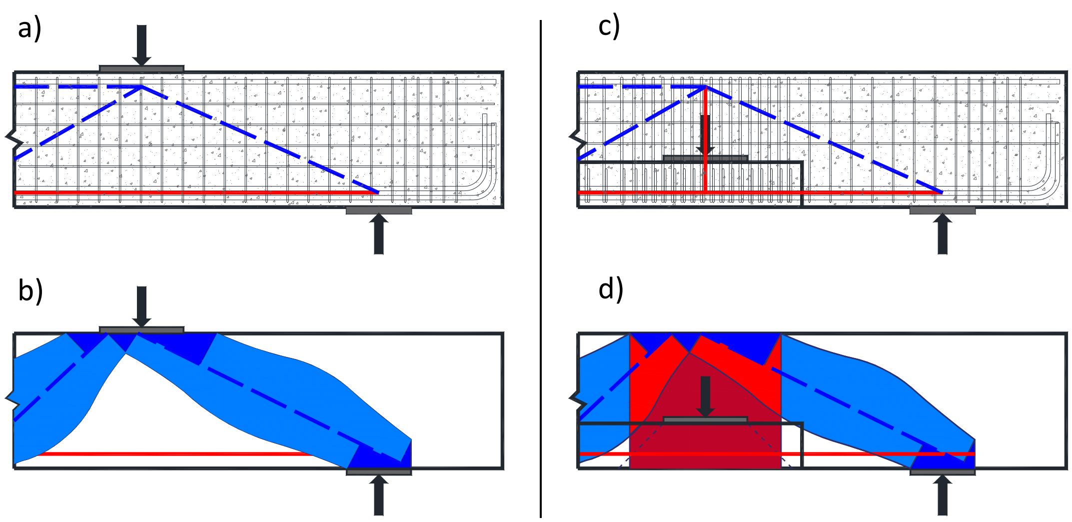

Figure 1 Top-chord loaded specimen: a) STM and b) *CCC* Node.
Bottom-chord loaded specimen: c) STM and d) *CCT* Node

RESEARCH SIGNIFICANCE

A review of previous literature revealed the scarcity of experimental investigation of Inverted-T specimens. Design criteria currently used in practice was based mostly on small-scale experiments. Therefore it was deemed necessary to investigate specimens of comparable size to field members in the current project. Findings from a small selection of the ongoing experimental work on full-scale specimens are presented in this paper for strength and serviceability performance. Ultimately the results of all the full scale Inverted-T investigation will be used to make Strut-and-Tie design recommendations.

EXPERIMENTAL WORK

Twenty tests have been conducted to the date on 10 full-scale bottom-chord loaded specimens with depths of 42 and 75 inches. Parameters varied in the tests were the overall depth, ledge depth and length, number of loading points, shear span-to-depth ratio, and reinforcement ratio. This paper focuses on the observed differences between test results of two bottom-chord loaded specimens from the current study (TxDOT Project 0-6416) and two top-chord loaded specimens from a previous study (TxDOT Project 0-5253) of comparable dimensions and reinforcing details.

TEST SETUP

Specimens were tested at the Ferguson Structural Engineering Laboratory at the University of Texas at Austin. The loading frame is an upside-down test setup for a simply supported beam. At each support six 3-in diameter steel rods threaded into the strong floor react against a 7,700-pound transfer beam. The beam is then pushed against a 2-in diameter roller sandwiched between two 2 inch thick steel plates under the transfer beams (Figure 2); a thin layer of self-leveling gypsum cement was applied between the steel plate and the beam to ensure a smooth planar bearing surface.

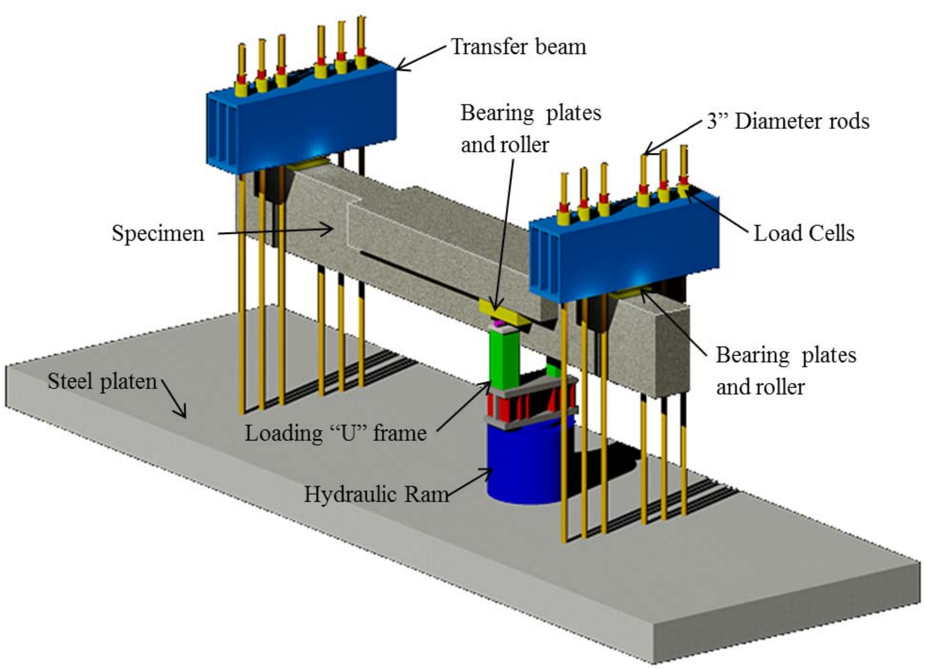

Figure 2 Test Setup. Upside-Down Simply Supported Beam.

Loads were applied using a 5-million pound capacity double-acting hydraulic ram for the beams with one point load and three 2-million pound capacity rams for the beam with multiple point loads. A "U" steel frame (Figure 2) was constructed to apply the loads to the ledges. The beam originally rested on steel frames at each end and was lifted by the application of load until it reacted against the transfer beams. Horizontal displacement and rotation was allowed with a 3-in diameter roller between two steel plates of 7 and 5 inches of thickness for the beams with one and three point loads respectively. A ¼-in reinforced neoprene bearing pad was placed between the steel plate and the concrete beam ensure a uniform load distribution avoiding stress concentrations.

Beams were monotonically loaded in 50-kip increments up to the appearance of the first diagonal crack, then in 100-kip increments up to failure. Crack comparator cards were used to measure the maximum shear crack widths after each load increment on both faces of the web by two researchers.

TEST SPECIMENS

Specimens were built using pre-mixed Class C concrete with design strength of 5 ksi. The mixture proportions are summarized in Table 1. Concrete cylinders were prepared conforming to ASTM C31 and tested according to ASTM C39 at 7 days, 28 days, and when the beam was loaded. Steel formwork was used to expedite the construction process and to ensure dimensional accuracy. Specimens were cured for 7 days and then stored indoors for at least 21 additional days before testing. Grade 60 deformed steel bars meeting the requirements of ASTM A615 were used. Coupons of each bar size were tested for each beam according to ASTM A370 to find the actual strength of the reinforcement as summarized in Table 2.

Table 1 Mixture proportions

Material	**Quantity**
Type I Portland Cement	387 lb/cy
Fly Ash	94 lb/cy
CA: 3/4" River Rock	1657 lb/cy
FA: Sand	1537 lb/cy
Water	22 gallons/cy
HRWR Admixture	19 oz/cy
Set Retardant Admixture	7 oz/cy
Water/Cement Ratio	0.48
Slump	7 inches

A summary of the details for the four beams analyzed in this paper are presented in Table 2. The first two specimens are part of the current study of bottom-chord loaded beams (TxDOT 0-6416); whereas the last two are top-chord loaded beams that were part of TxDOT Project 0-5253.

Table 2 Specimen summary

Specimen	b_w	b_{le}	h	h_{le}	d	d'_{le}	ρ_l	ρ_l'	ρ_v	Size and Spacing (s_v)	ρ_h	Size and Spacing (s_h)	ρ_{ha}	Size and Spacing (s_{ha})	Support plate	Load plate	a/d
DS1-42-1.85-03	21	42	42	0.5 h	37.6	18.7	0.024	0.006	0.003	# 4 @ 6.5"	0.003	# 4 @ 6.5"	0.012	# 6 @ 3.5	16 x 20	26 x 9	1.96
SS3-42-1.85-03	21	42	42	0.33 h	37.6	11.7	0.024	0.006	0.003	# 4 @ 6.5"	0.003	# 4 @ 6.5"	0.014	# 6 @ 3.0	16 x 20	18 x 9	1.85
I-03-2[1]	21	*	44	*	38.5	*	0.023	0.012	0.003	# 4 @ 6.5"	0.003	# 4 @ 5.75"	*	*	16 x 21	20 x 21	1.84
III-1.85-03b[1]	21	*	42	*	38.6	*	0.023	0.012	0.003	# 4 @ 6.0"	0.003	# 5 @ 10.1"	*	*	16 x 21	20 x 21	1.84

[1] top-chord loaded specimens from project 0-5253
*Note: Values marked as * are not applicable for rectangular specimens*
All variables are defined in the notation section.

Reinforcement details and cross sections are shown in Figure 3 and Figure 4. The nomenclature for the two bottom-chord loaded specimens is as follows: The first character refers to the depth of the ledge, Deep (h_{le} = 0.5 h) or Shallow (h_{le} = 0.33 h). The second character (S) refers to the length of the ledge; in this case both specimens have short ledges (i.e. ledges extend a distance h_{le} beyond the end of the loading plate in the longitudinal direction). The third character refers to the number of pairs of point loads applied to the specimen. The next two characters refer to the overall depth of the specimens; in this case both specimens have a depth of 42 inches. The next number refers to the shear span-to-depth ratio (1.85 for the two specimens). The final number represents the ratio of shear reinforcement to effective area; in this case both specimens have a ratio of 0.3%.

The four specimens presented in this paper were selected for their similarities to allow for better comparisons. The first two specimens have the same characteristics; except that the first one has a deep ledge and one load point and the second one has a shallow ledge and three point loads.

The last two specimens in Table 2 have the same characteristics as the first two, except that they were loaded at the top-chord with slightly different bearing plate sizes. All specimens had the same web area and shear reinforcement to effective area ratio (ρ_v and ρ_h), achieved through slightly different bar sizes and spacing.

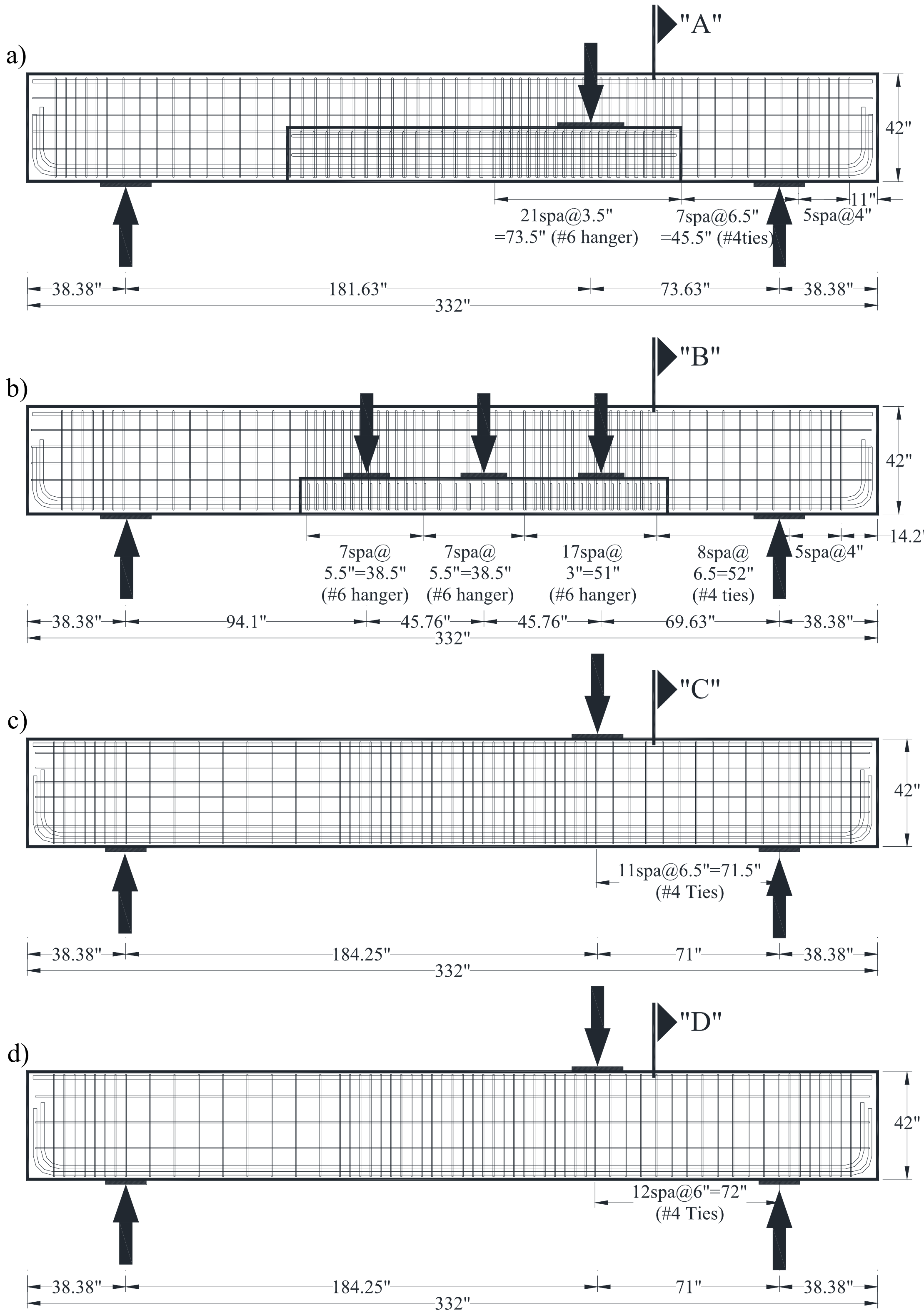

Figure 3 Specimen details;
a) DS1-42-1.85-03, b) SS3-42-1.85-03, c) I-03-2[1], d) III-1.85-03b[1]

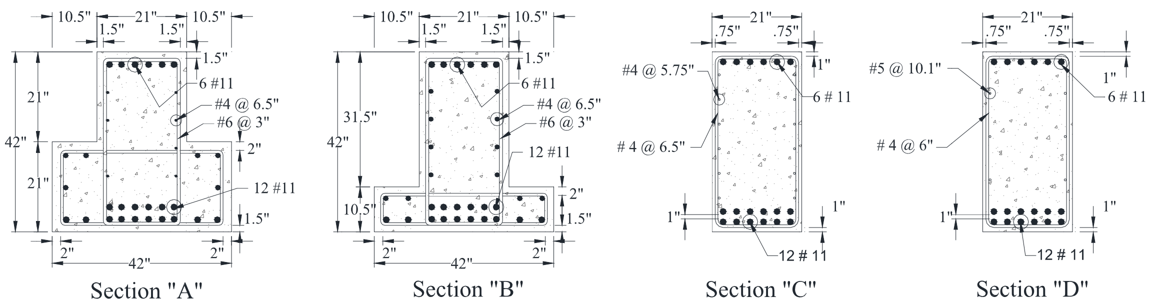

Figure 4 Cross-sections at loading points, a) DS1-42-1.85-03, b) SS3-42-1.85-03, c) I-03-2[1], d) III-1.85-03b[1]

INSTRUMENTATION

Longitudinal and transverse steel strains were monitored during the tests using internal strain gages. Reactions were measured using 12 load cells with a capacity of 500 kips each (refer to Figure 2). The applied loads were measured using a pressure transducer attached to the hydraulic pump feeding the loading rams. Deflections were measured using five linear potentiometers (refer to Figure 5), a typical arrangement of linear potentiometers included: one at each support location (to filter rigid body movements), two at the location of the nearest load to the support to check symmetry of the applied loads (i.e. no torsion), and one at midspan for the beams with one point load or at the location of the mid-load for the beam with three point loads (to obtain the beam deflection with respect to the supports).

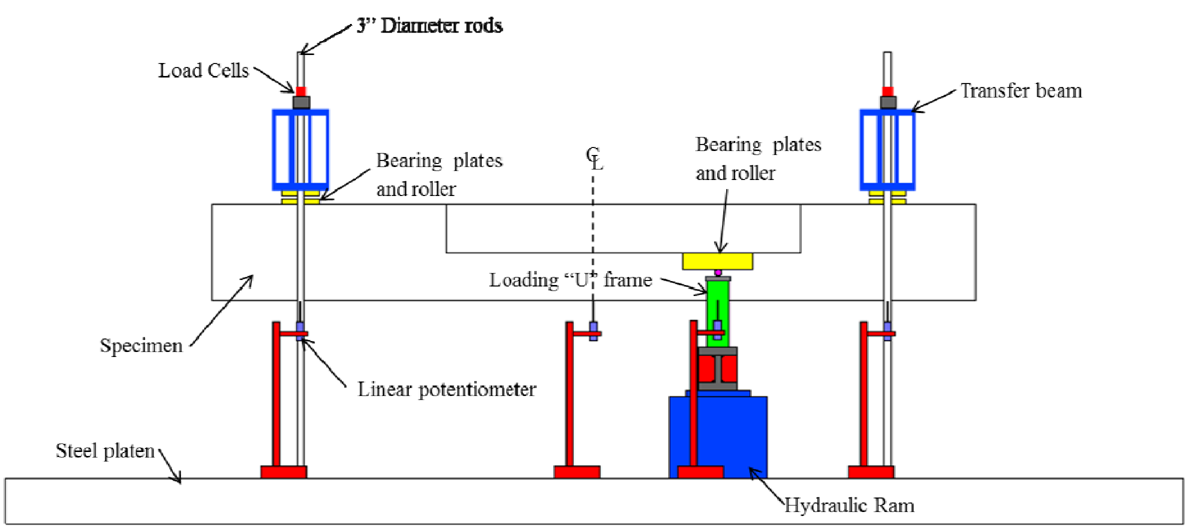

Figure 5 Elevation of test setup

EXPERIMENTAL RESULTS

A summary of the strength and serviceability data collected during the tests is presented in Table 3. One of the objectives of the current study was to evaluate the differences between top- and bottom-chord loaded members. In order to analyze the implications of the tension fields induced near the loading points in bottom-chord loaded specimens the results from the present study (first two specimens) were compared with specimens of a previous study[1] of top-chord loaded members (last two specimens).

Table 3 Summary of experimental results

Specimen	b_w	d	f'c	fy_l	fy_{sh}	fy_{ha}	a/d	V_{crack}	V_{test}	$\frac{V_{crack}}{V_{test}}$	$\frac{V_{crack}}{\sqrt{f'c}\, b_w d}$	$\frac{V_{test}}{f'c\, b_w d}$
DS1-42-1.85-03	21	37.6	5258	69.2	63.1	63.7	1.96	172	712	0.24	3.0	0.17
SS3-42-1.85-03	21	37.6	5891	68.6	67.3	64.7	1.85	126	523	0.24	2.1	0.11
I-03-2[1]	21	38.5	5240	73	67	*	1.84	144	569	0.25	2.5	0.13
III-1.85-03b[1]	21	38.6	3300	69	64.5	*	1.84	114	471	0.24	2.4	0.18

[1] top-chord loaded specimens from project 0-5253
*Note: Values marked as * are not applicable for rectangular specimens*
All variables are defined in the notation section.

Strength values are normalized by the effective area (i.e. $b_w d$) and by the strength of concrete, since it is associated with the ultimate capacity of the specimens. For the case of serviceability values, they are normalized by the effective area and $\sqrt{f'_c}$ as the first cracking of the beam is associated with the tensile strength of the concrete.

STRENGTH RESULTS

The strength of the specimens is reported as V_{test}, the shear at the critical section at the maximum applied load. The critical section was defined as the midpoint of the shear span, at a distance a/2 from the center of the support and the calculations to obtain V_{test} are presented in Figure 6.

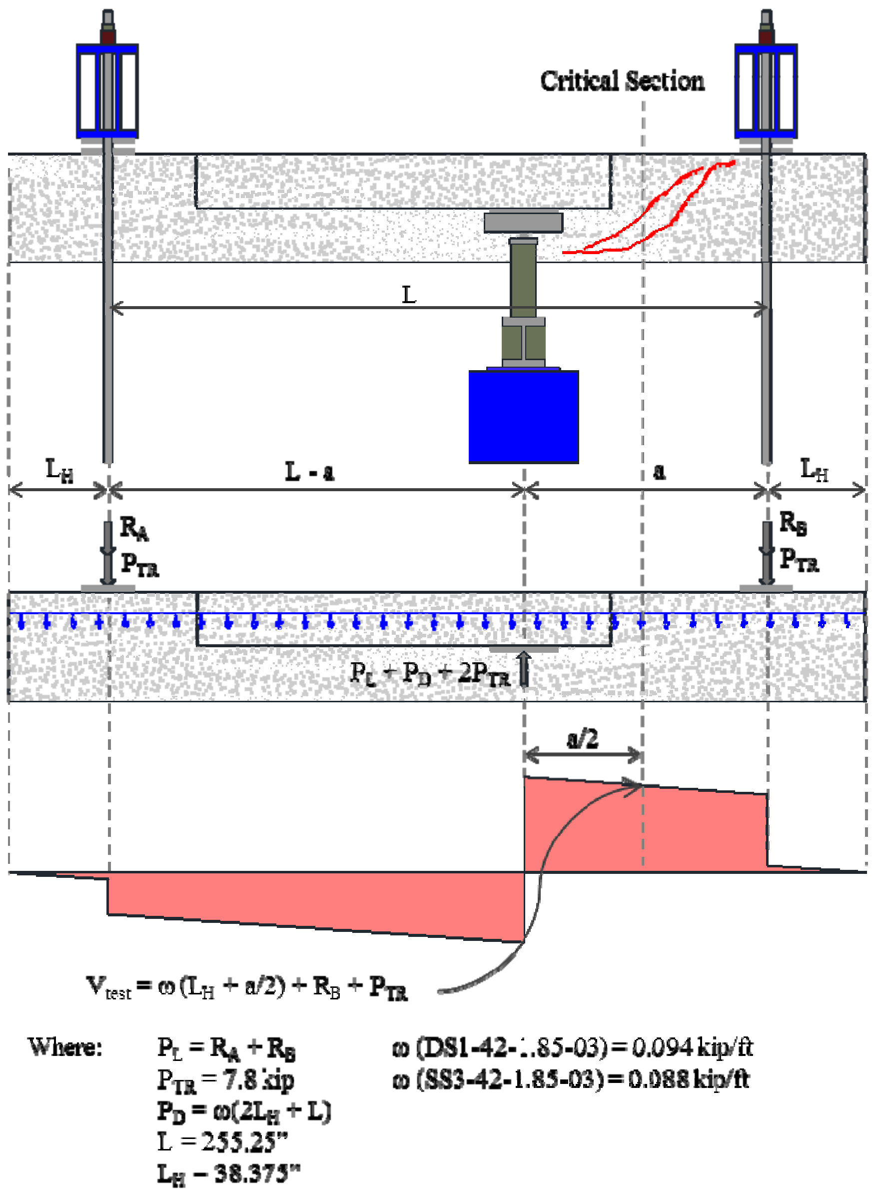

Figure 6 Free-body and shear force diagram for one point load specimens[2]

Strength results can be observed in Figure 7. Results show a wide range of scatter, typical of shear tests. However, there is a significant difference between the specimen loaded at midheight and the one loaded at one third of its height; suggesting that loading the beam below its middepth decreases its ultimate capacity. This may be due, at least partially, to the fact that with a shallow ledge the load has a limited area to spread before being "hung" up to the compression chord. The resulting tension field is concentrated in a narrower area in the beam with a shallow ledge than in the beam with a deep ledge. Similar observations were made by Ferguson in a previous study that investigated the effects of loading conditions on shear capacity.[3]

The two bottom-chord loaded specimens were loaded with either one or three point loads. Additional tests investigating the effects of the number of load points are planned and will aid in drawing final conclusions.

The last two specimens were loaded at their compression chord and have similar geometry, reinforcement ratios and shear span-to-depth ratio to the first two specimens. The ultimate strength of the last specimen was found to be approximately 40% higher than that of the third specimen.

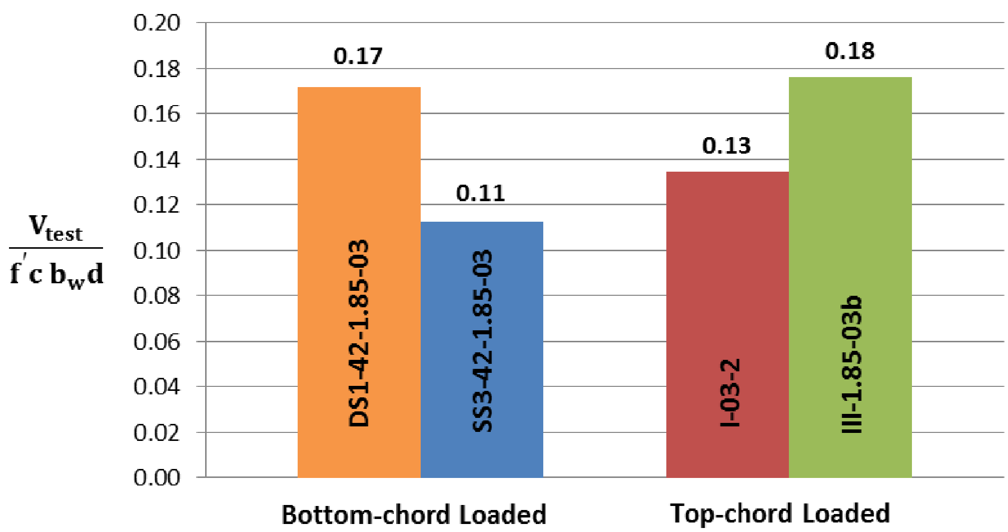

Figure 7 Ultimate strength for bottom- and top-chord loaded members

Pictures of the specimens at failure are shown in Figure 8 for bottom-chord loaded specimens and in Figure 9 for top-chord loaded specimens. The four specimens presented similar cracking patterns. The specimens failed in shear after crushing of concrete in the strut and nodal regions occurred.

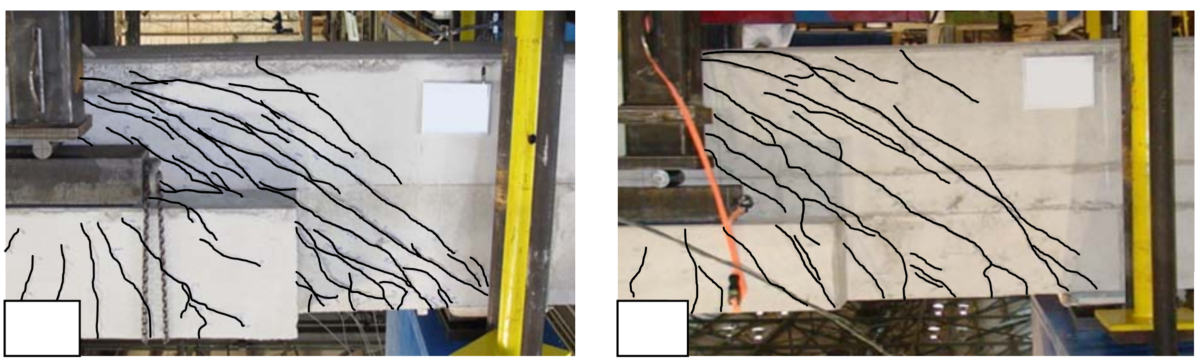

Figure 8 Inverted-T specimens: a) DS1-42-1.85-03 and b) SS3-42-1.85-03 at failure

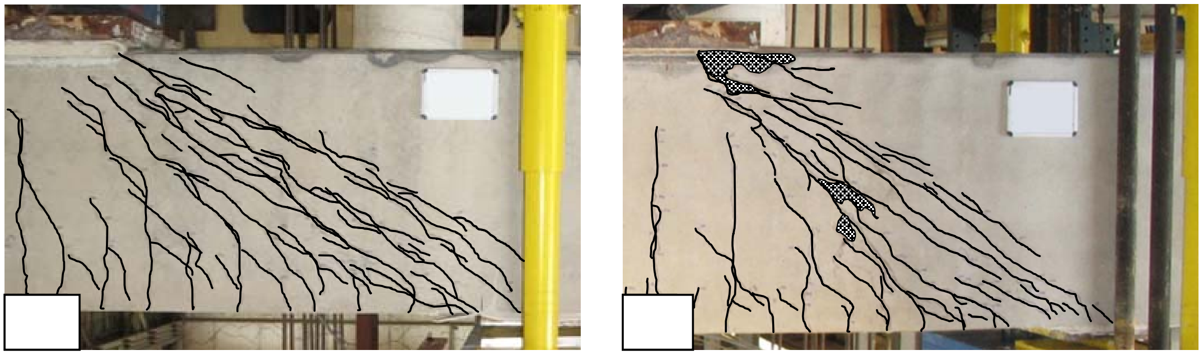

Figure 9 Deep beam specimens: a) I-03-2 and b) III-1.85-03b at failure

SERVICEABILITY RESULTS

Two aspects were considered when evaluating the serviceability performance of the specimens: (1), the shear carried by the beam at the moment of the formation of the first diagonal cracking (Figure 10), and (2) the progression of the crack widths with respect to the percentage of the ultimate load carried by the beam (Figure 11).

In order to identify the value of V_{crack} strain gage readings were analyzed. The load at which the first diagonal crack appeared is commonly accompanied by a sudden jump in the strain readings of the stirrups in that location. This value was verified by visual observations between the load increments.

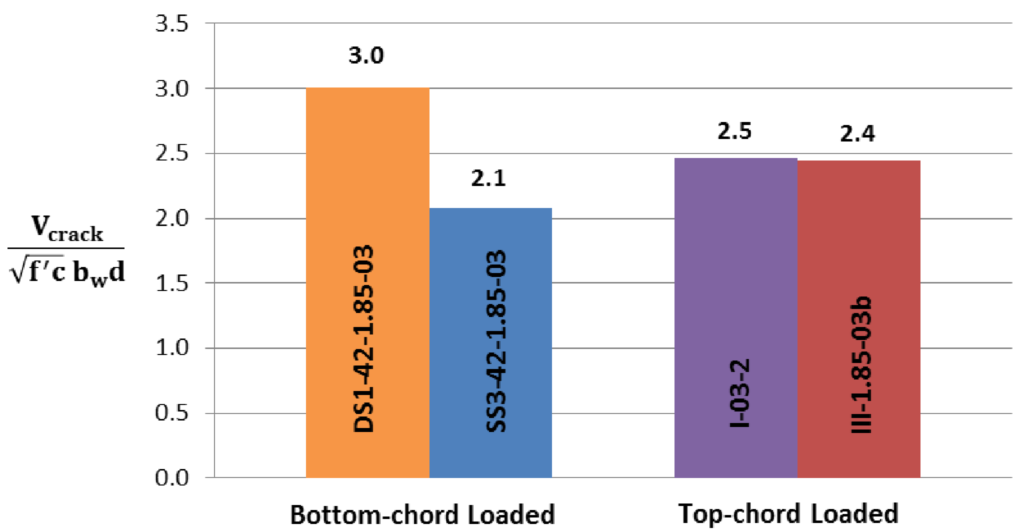

Figure 10 Cracking shear for bottom- and top-chord loaded members

A significant decrease in the cracking shear for the Inverted-T beam with shallow ledges was observed compared with that of the beam with deep ledges. This decrease could be related to the concentration of the tension field in a narrower area in beams with shallow ledges. The cracking shear of the two top-chord loaded specimens showed no significant difference regardless of the differences in the size and spacing of the stirrups (all four specimens have a shear reinforcement ratio of approximately 0.003 in both directions). However, comparing top- versus bottom-chord loaded specimens, the cracking shear was observed to be higher for the case of deep ledges, and lower for the case of shallow ledges. Further investigations are needed to corroborate these observed trends.

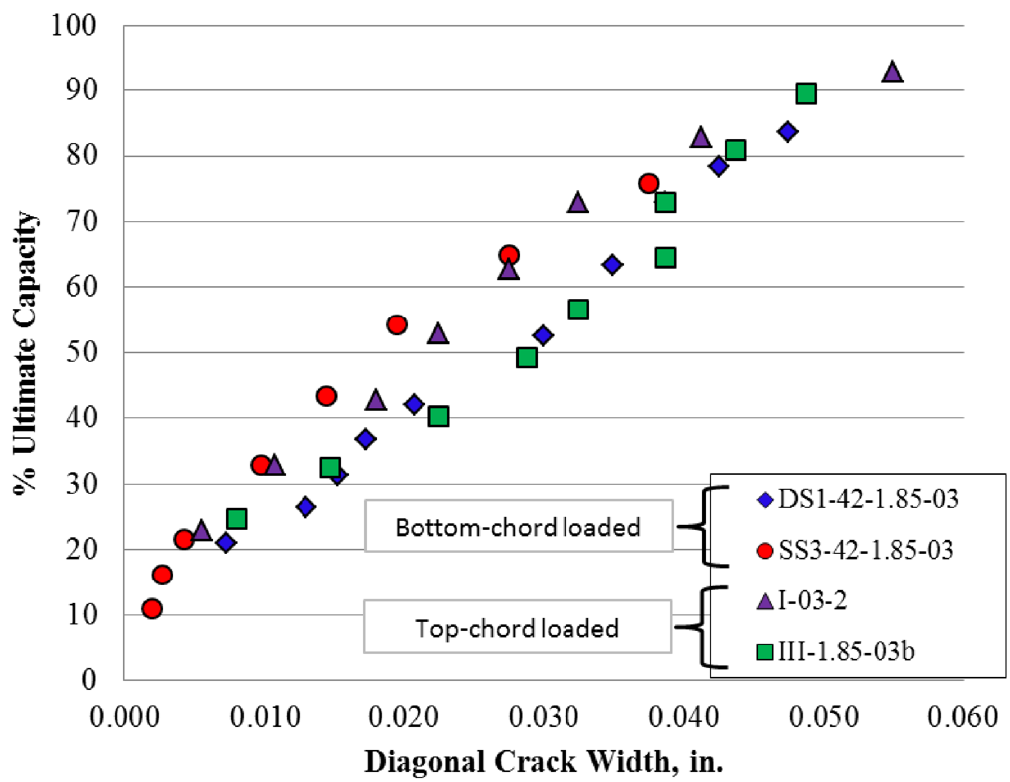

Figure 11 Diagonal crack widths versus percent ultimate capacity for top- and bottom-chord loaded specimens

It can be observed in Figure 11 that there is a strong relationship between applied loads and crack widths. There is no clear difference between crack widths of top- and bottom-chord loaded specimens. The four specimens follow the same trend, and for the same crack width the specimens have a difference in applied loads no larger than approximately 15%.

Comparing the two bottom-chord loaded specimens it can be observed that the beam with shallower ledge (h_{le}=0.33h) presented earlier web cracking at 11% of its ultimate capacity. In contrast, the specimen with deeper ledge (h_{le}=0.5h) presented the first diagonal cracking at 21% of its ultimate capacity.

The two top-chord loaded specimens had approximately the same shear reinforcement ratio, using No. 4 bars spaced at 6.5 and 6.0 inches for the stirrups, and No. 4 at 6.5 inches and No. 5 at 10.1 inches respectively in the horizontal direction. We can observe that the specimen with smaller bars but closer spacing presented narrower cracks showing a better serviceability performance. However, both beams presented approximately the same cracking shear and ultimate capacity.

It is also worth noting that for the same crack width, the beam with the shallower ledge is closer to failure than the beam with the deeper ledge. This observation holds true up to about 75% of the ultimate capacity; however, above this value both beams presented similar crack widths for the same percentage of ultimate capacity.

APPLICABILITY OF DESIGN PROVISION OF TxDOT PROJECT 0-5253 TO BOTTOM-CHORD LOADED MEMBERS.

A new Strut-and-Tie Modeling (STM) design procedure was developed by the previous TxDOT Project 0-5253 to estimate strength and serviceability performance of deep beams. Modifications to ACI 318-08 and AASHTO LRFD (2008) code provisions for STM were proposed.

The principal Strut-and-Tie Modeling provisions proposed by TxDOT Project 0-5253 were:

- Use STM to design sections with shear span-to-depth rations smaller than 2.
- Bond stresses resulting from the force in a developed tension tie need not to be applied as a concentrated force to the back face of the *CCT* nodes (where C represents compression and T tension)
- Smeared nodes are not as critical as singular nodes and do not need to be checked.
- Capacities of all faces of *CCC* and *CCT* nodal regions can be increased by a factor $\sqrt{A_2/A_1}$ when triaxial confinement is present.
- A minimum of 0.3% of shear reinforcement evenly spaced in each orthogonal direction should be provided in the effective strut areas. Maximum spacing should be limited to the smaller of d/4 or 12in.
- Concrete efficiency factors shall be taken as:

  0.85 bearing and back face of *CCC* nodes

  0.70 bearing and back face of *CCT* nodes

  0.85 - $f'c/_{20ksi}$ at *CCC* and *CCT* strut-to-node interfaces. But no more than 0.65 or less than 0.45.

  0.45 at *CCC* and *CCT* strut-to-node interface for structures that do not contain crack control reinforcement.

These provisions were calibrated using top-chord loaded members and were used to calculate ultimate strength predictions for the bottom-chord loaded specimens reported in this paper in order to validate their applicability or to identify any required modifications. A summary of the actual to predicted capacity ratios is shown in Figure 12. Ratios larger than or equal to 1.0 would be considered conservative estimates of the shear capacity of the beam.

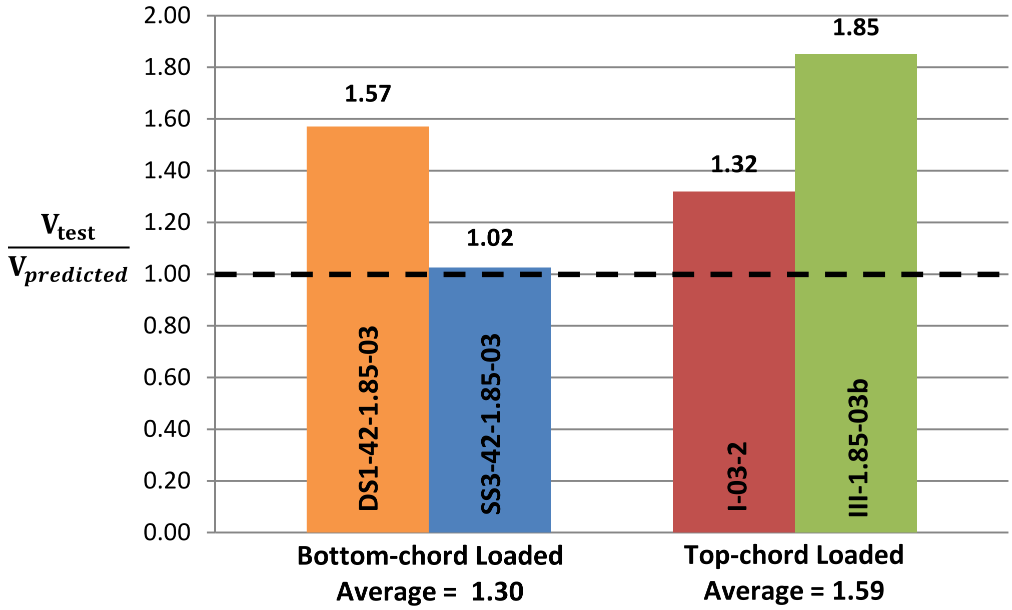

Figure 12 Actual to predicted capacity ratios

The actual to predicted capacity ratios for the two bottom-chord loaded specimens are greater than 1.0, which is indicative of a conservative design. However, we can observe that the average of the bottom-chord loaded specimens is lower than the top-chord loaded specimens. This indicates that although the design is safe, the conservatism decreased by roughly 20%. Additional test results will help to verify these observations.

SUMMARY

Several recently built Inverted-T caps in Texas showed significant inclined cracking triggering concern about current design procedures for such caps. Some of the objectives of the present study are to evaluate the differences between tension- and compression-chord loaded members to obtain a better understanding of the structural behavior of Inverted-T straddle bent caps, and to develop design recommendations to prevent or minimize such cracking in the future.

Findings pertaining to the strength and serviceability implications of web tension fields were briefly discussed. A significant difference was observed between specimens loaded at midheight and specimens loaded at the bottom third of their height. This observation suggests that loading the beam below its middepth decreases its ultimate capacity.

There is a significant decrease in the shear carried at the appearance of first diagonal cracking for the beam with shallow ledges compared with the specimen with deep ledges. A strong relationship between applied loads and crack widths was observed. For the same crack width, the specimens have a difference in applied loads no larger than approximately 15%.

A recent study[1] of top-chord loaded specimens developed design guidelines using Strut-and-Tie modeling for strength and serviceability of deep beams; results of the current study are compared with those of the previous TxDOT Project 0-5253 to evaluate the applicability of the new design provisions to Inverted-T beams or to identify any required modifications.

The experiments presented in this paper suggest that the Strut-and-Tie Modeling (STM) design provisions proposed by the previous TxDOT Project 0-5253 produce safe estimates of ultimate strengths of bottom-chord loaded specimens. The conservatism observed in bottom-chord loaded members was lower than that observed in top-chord loaded members. This is especially true for beams with shallow ledges where the tension field is concentrated in a narrower area.

This paper contains a small selection of tests from a larger ongoing research study of Inverted-T straddle bent caps (TxDOT Project 0-6416). Results show the wide range of scatter typical of shear tests due to the natural variability in shear behavior. A larger number of tests is required to further validate observations and discover additional trends. The test program will be completed with over 20 specimens to complement the results presented in this paper before drawing the final conclusions.

ACKNOWLEDGEMENTS

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DISCLAIMER

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NOTATION

a	=	shear span, ft
a/d	=	shear span-to-depth ratio
A1	=	loaded area
A2	=	effective loaded area
b_{le}	=	ledge width, in
b_w	=	web width, in
CCC	=	nodes in which only compressive struts intersect
CCT	=	nodes anchoring one tension tie
d	=	distance form extreme compression fiber to centroid of tensile reinforcement, in
d'_{ledge}	=	distance from extreme compression fiber to centroid of tensile reinforcement of ledge, in
fy_l	=	tensile strength of tensile reinforcement, ksi
fy_{ha}	=	tensile strength of hanger reinforcement, ksi
fy_{sh}	=	tensile strength of shear reinforcement (A_v and A_h), ksi
f'c	=	compressive strength of concrete cylinders at the day of the test, psi
h	=	beam height, in
h_{le}	=	ledge height, in
L	=	clear span, in
L_H	=	overhang length, in
P_D	=	weight of test specimen, kip
P_L	=	load applied by rams, kip
P_{TR}	=	weight of each transfer girder, kip
P_U	=	ultimate load carried by the specimen, kip
R_a	=	Reaction measured by the load cells of support a, ksi
R_b	=	Reaction measured by the load cells of support b, ksi
s_v	=	spacing of vertical shear reinforcement, in
s_h	=	spacing of horizontal shear reinforcement, in
s_{ha}	=	spacing of hanger reinforcement, in
V_{crack}	=	shear carried in the test region when the first diagonal crack formed, kip
V_{test}	=	maximum shear carried in test region, including the estimated self-weight of the specimen and transfer girders, kip
ω	=	specimen self-weight, kip/ft
ρ_l	=	ratio of longitudinal tensile reinforcement to effective area ($As / b_w d$)
ρ_l'	=	ratio of longitudinal compression reinforcement to effective area ($A's / b_{le} d$)
ρ_h	=	ratio of horizontal shear reinforcement to effective area ($A_h / b_w s_h$)
ρ_{ha}	=	ratio of hanger reinforcement to effective area ($A_{ha} / b_w s_{ha}$)
ρ_v	=	ratio of vertical shear reinforcement to effective area ($Av / b_w s_v$)
Load Plate	=	dimensions of the load bearing plate measured in the longitudinal

and transverse direction of the beam (*l x w*), in

Support Plate= dimensions of the support bearing plate measured in the longitudinal and transverse direction of the beam (*l x w*), in