Staff Bridge Branch

ACCELERATED BRIDGE CONSTRUCTION UTILIZING PRECAST PIER CAPS ON STATE HIGHWAY 69 OVER TURKEY CREEK, HUERFANO COUNTY, CO

IBRD RESEARCH PROJECT 102470

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16. Abstract

The purpose of this report is to document Accelerated Bridge Construction (ABC) techniques on IBRD (Innovative Bridge Research and Development) project 102470 for the construction of Bridge N-16-Q on State Highway 69 over Turkey Creek. The construction work was done on project FBR 069A-022, PC 15772, and was completed in July of 2011. During the course of the project, participants were able to:

- develop a design methodology,

- develop universal details, and

- demonstrate how fast the bridge could be constructed using this technique.

The project demonstrated faster pier erection by utilizing precast pier caps to eliminate concrete cure time from the critical path in the construction schedule.

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IBRD Research Project 102470

for

Accelerated Bridge Construction Utilizing Precast Pier Caps

State Highway 69 over Turkey Creek
Huerfano County, Colorado

Richard L. Osmun
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TABLE OF CONTENTS

TABLE OF ILLUSTRATIONS

OVERVIEW

The purpose of this report is to document Accelerated Bridge Construction (ABC) techniques on IBRD (Innovative Bridge Research and Development) project 102470 for the construction of Bridge N-16-Q on State Highway 69 over Turkey Creek. The construction work was done on project FBR 069A-022, PC 15772, and was completed in July of 2011. During the course of the project, participants were able to:

- develop a design methodology,

- develop universal details, and

- demonstrate how fast the bridge could be constructed using this technique.

The project demonstrated faster pier erection by utilizing precast pier caps to eliminate concrete cure time from the critical path in the construction schedule.

PROJECT DESCRIPTION

This research project demonstrated an Accelerated Bridge Construction (ABC) technique for installing precast pier caps that are permanently fixed to the superstructure and pinned to the tops of the individual columns. The project involved replacement of existing structure N-16-L that was built in 1923 with N-16-Q on a short re-alignment of SH 69. Figure 1 shows the location of the bridge on SH-69 at mile marker 18.94, about 19 miles northwest of I-25 at Walsenburg in south-central Colorado.
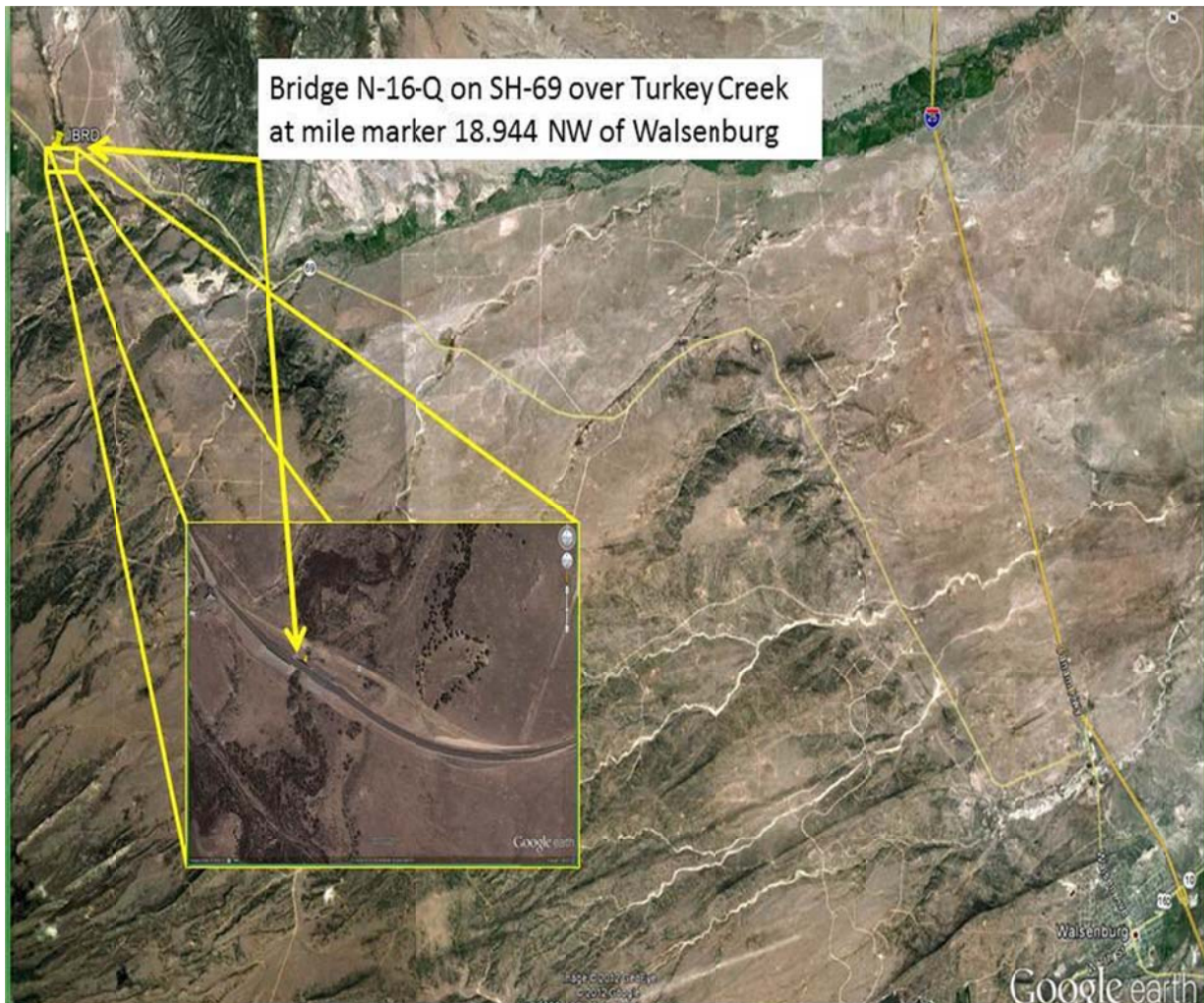
Figure 1 - SH 69 mm 18.9 Northwest of Walsenburg

CONCEPT

With this ABC process, crane(s) can be repositioned to begin setting girders as soon as a pair of precast pier caps is set. Temporary stays are used to fix the precast pier cap to the individual columns in a bent as shown in Figure 2. After the deck has been placed and has reached its full design strength, the precast pier cap will be fixed to the superstructure. The temporary stays can then be removed leaving a series of pinned joints between the precast pier cap and the supporting columns.

The width of the precast pier cap matches the width of the column cap to facilitate the installation of temporary stays *i* and *j,* as shown in Figure 2. After the disc bearings have been installed, the precast pier cap is erected using rigging like that shown in Figures 3 and 4. Variable length slings are required to plumb the cap as it is set preventing the temporary stays from binding when they are installed. Temporary stays *i* and *j* are installed while the crane(s) are still in place holding the pier cap to prevent it from rotating. With the temporary stays in place, the crane(s) can be unhooked from the cap and repositioned to begin erecting girders in span I.
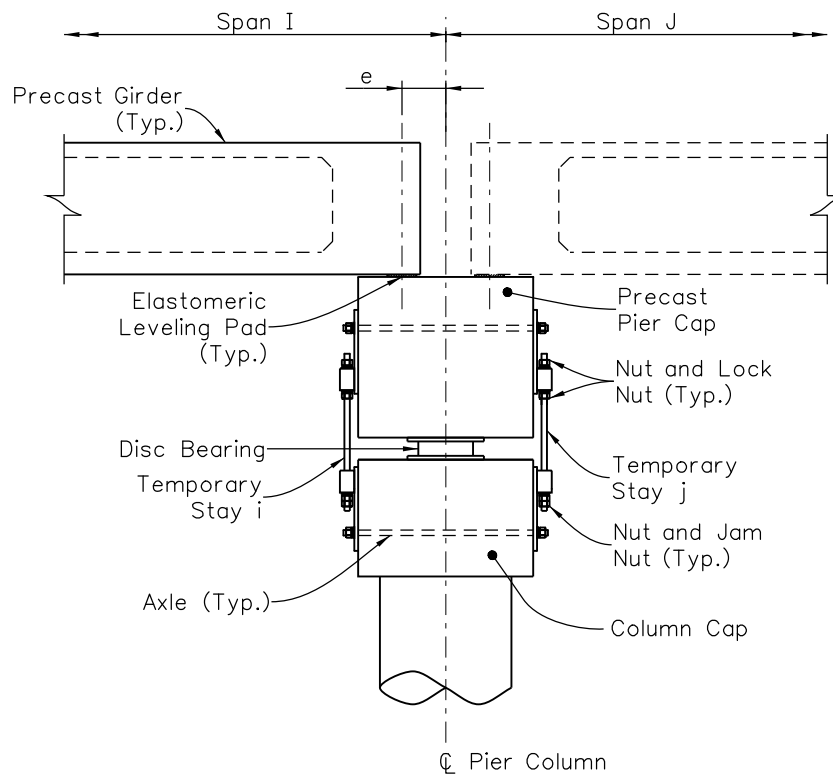
Figure 2 - **Typical Pier Section**

3
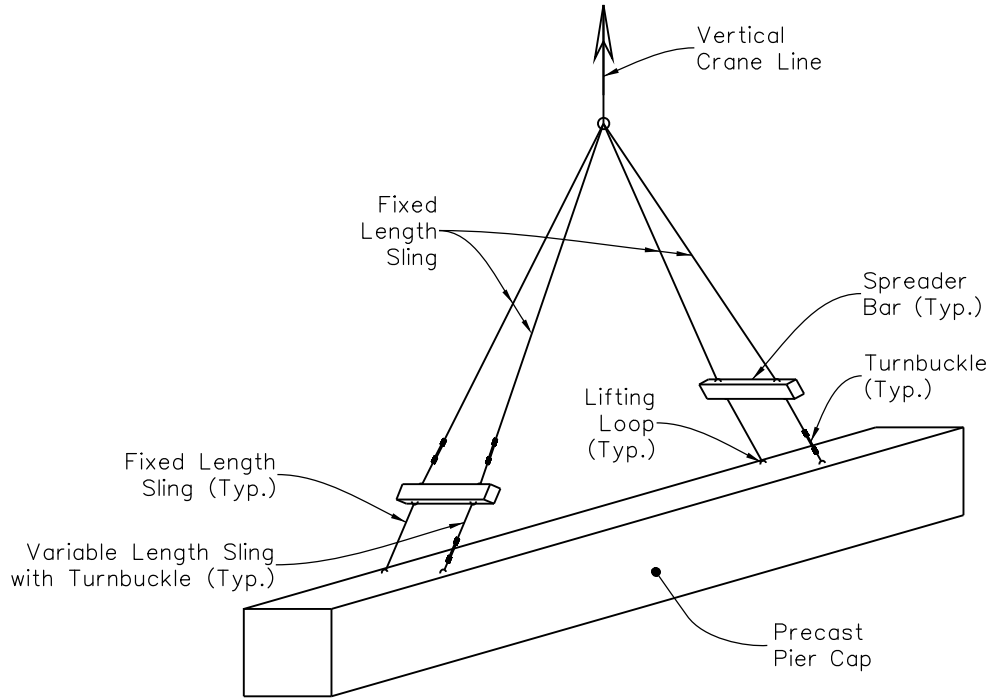
Figure 3 - Short Cap / One Crane
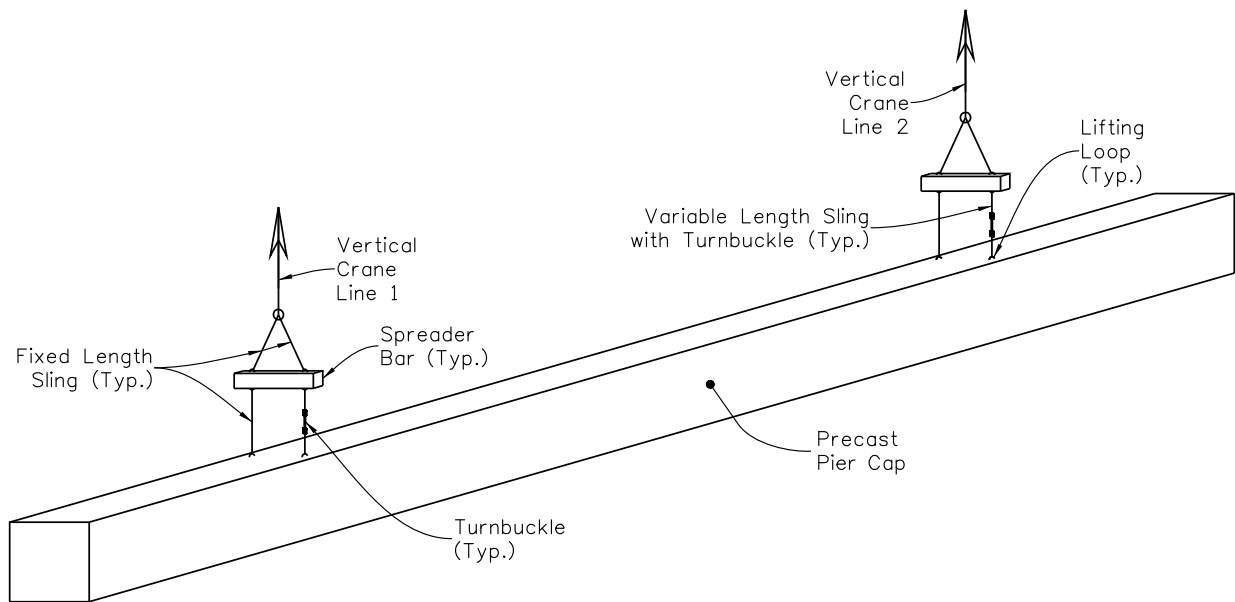
Figure 4 - Long Cap / Two Cranes

4

DEMONSTRATION PROJECT DESCRIPTION

A single span bridge with precast box girders, as shown in Figure 5, was chosen to demonstrate the ABC concept. This bridge was chosen over some of the other candidates because it was available and required only six disc bearings (three per abutment). A comparison between Figure 2 and Figure 6 for pier and abutment cap details shows that the stays in both caps function identically making it possible to minimize research and construction expenses for this project without altering the concept for this research.
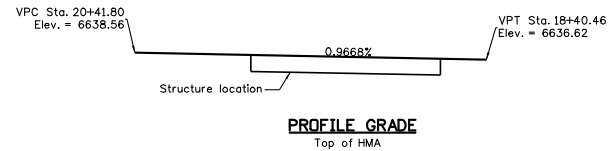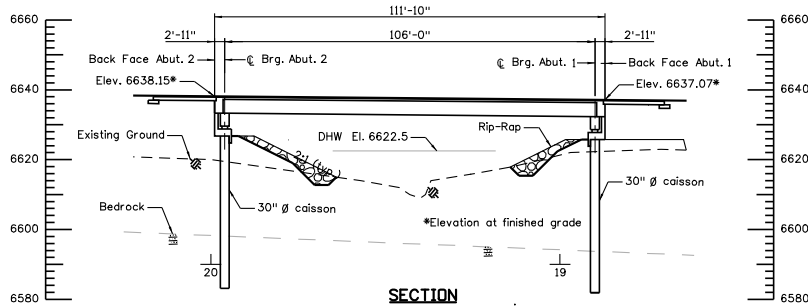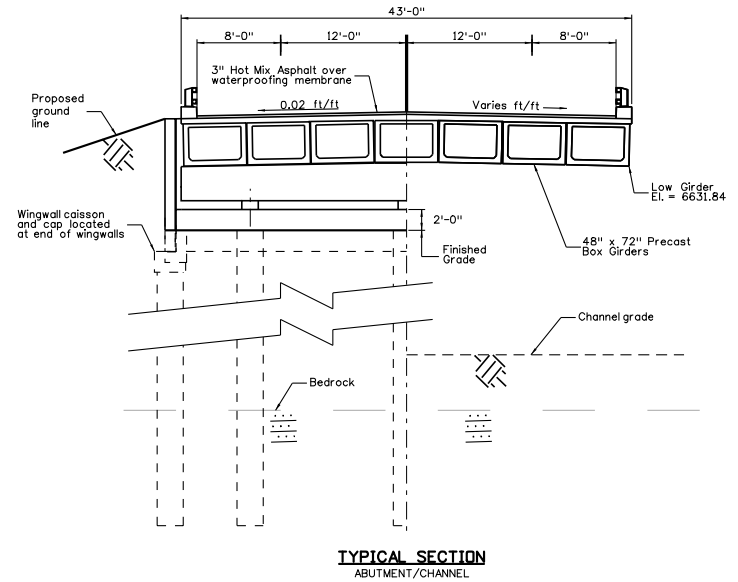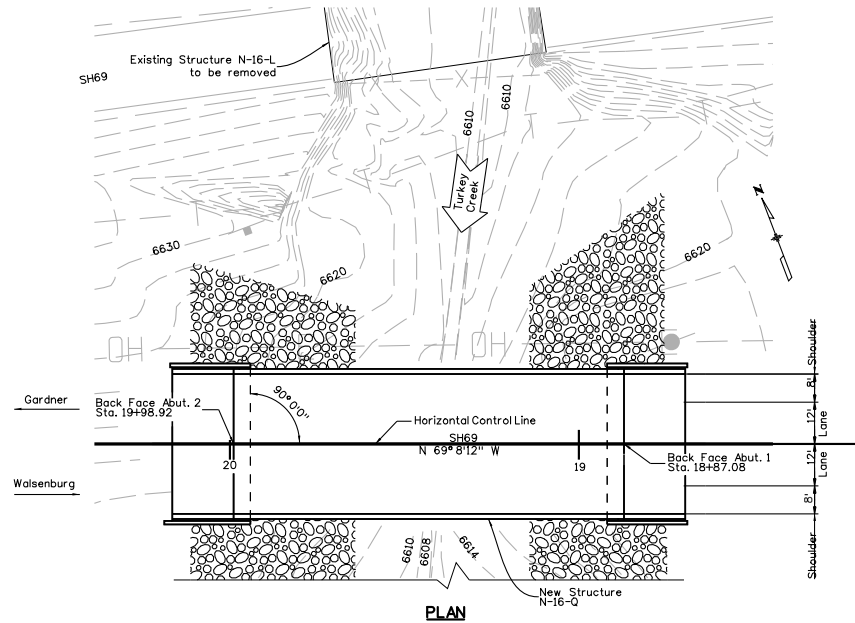
Figure 5 - General Layout N-16-Q

6

An 11.5" eccentricity was built into the abutment geometry as shown in Figure 6 to replicate the eccentricity *e*, as shown in Figure 2. This puts the back stays in tension as the front stays go slack. The front stay in Figure 6 represents stays *i* and *j* in Figure 2 which act as counters. For example, when girders are being erected in span *I* (Figure 2), the *j* temporary stays prevent the cap beam from rotating counterclockwise due to eccentricity *e* as the *i* temporary stays become slack.
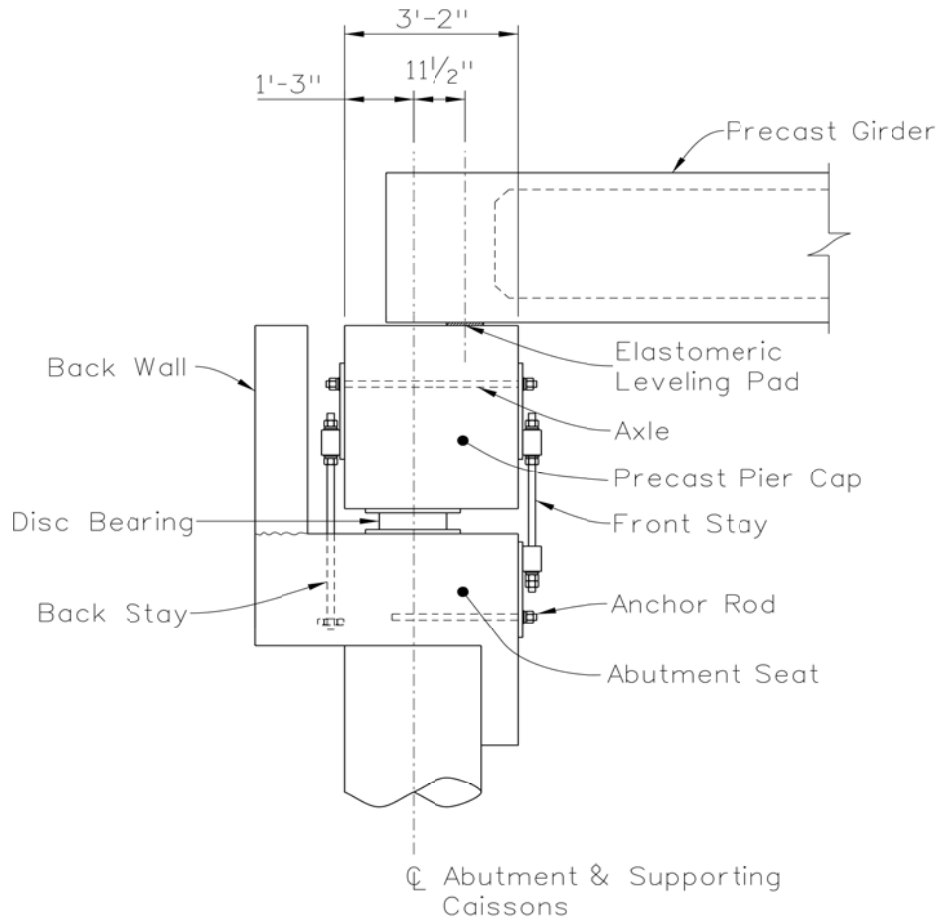
Figure 6 - Typical Abutment Section

The ***back stay*** in Figure 6 is a one-of-a-kind feature used only for single span structures like this one. Since the back stays were cast into the abutment seat, they could not be removed after the deck was poured. To free the pier cap on its bearings, the back stays were torch-cut after the deck concrete reached 80% of its design strength.

INSTALLATION OF BEARINGS

Using normal construction tolerances, the bearings must be installed at the specified locations and to the correct elevations as shown in Figure 7. During initial construction, anchor bolts are set in the abutment seat (or column cap for typical construction) at the bearing locations using a steel template to ensure correct spacing and alignment.

The bearings are installed on top of a welded shim stack that is used to (1) eliminate bending in the masonry plate and (2) position the top of the upper bearing plate to the required elevation $\pm 1/16$". To position the shim stack correctly under the bearings' masonry plates, a plywood keeper, thinner than the shim stack, fits over the anchor bolts. The shim stack fits snugly in a cut-out which temporarily secures the shim stack in place while the bearings are positioned and until the cap beam has been installed and the crane released. The weight of the pier cap will keep the welded shim stack in place allowing the plywood template to be removed to accommodate the grouting operation. The grout protects the anchor bolts and the welded steel shim stack from corrosion caused by standing water, snow and/or ice.

TYPICAL SECTION
(Bearing Type III)

℄ Abutment

** Shear pin

**,* Polytron disc

* Upper bearing plate
b x 1'-4½" x 1'-4½"

* Lower bearing plate
a x 1'-4½" x 1'-4½"

Embedment plate
see detail

Precast cap

6½"

10"

Abutment seat

Masonry plate
see detail

Welded shim stack
see detail

Thin set grout

⅜ 6
⅜ 6

Centered on front, left, and right
sides of upper bearing plate (Typ.).
Upper bearing plate temperature
next to disc must not exceed
225°F during welding.

Temporary ¾" plywood keeper,
see detail this sheet and Notes
2.a. and 2.g. on Dwg B11.

1½" ∅ x 2'-0" threaded anchor rod with lock
and jam nuts and hardened washers, 1'-4"
embedment into concrete seat, set with steel
template (Typ.) (Contractor's responsibility)

1" Typ.

WELDED SHIM STACK DETAIL
(Contractor's responsibility)

1'-4½"
℄
¼
Seal weld

Place thin shims
(¹⁄₁₆" min.) between
thick shims prior to
welding

Thick shims
(½" min.)

EMBEDMENT PLATE DETAIL
(Precast Fabricator's responsibility)

℄ ⅞" ∅ x 6" shear
studs each side (total 4)

3"   1'-8"   3"

2'-2"
Typ.

NOTES TO BEARING MANUFACTURER:

MAXIMUM EXTREME EVENT LATERAL LOAD AFTER CONSTRUCTION = 0.27(450K DL + 0.5 X 167K LL) = 144 KIPS.
GRADE 50 STEEL, A325 BOLTS, ASTM F436 HARDENED STEEL WASHERS.
EVERYTHING TO BE SUPPLIED BY BEARING MANUFACTURER U.O.N.
* THICKNESS AS PER MANUFACTURER (ASSUMED a = 2.5", b = 1.5" & DISC THICKNESS = 1.25" FOR DESIGN).
** DIAMETERS AS PER MANUFACTURER (ASSUMED 4" FOR SHEAR PIN AND 12¼" FOR POLYTRON DISC FOR DESIGN).

LIMIT STATE	DURING CONSTRUCTION		AFTER CONSTRUCTION	
	MAX. AXIAL LOAD (KIP)	ASSOCIATED ROTATION (RAD.)	MAX. AXIAL LOAD (KIP)	ASSOCIATED ROTATION (RAD.)
SERVICE I	495	N/A	521	N/A
STRENGTH I	634	0.0192	743	0.023

INFORMATION TABLE FOR TEMPORARY STAYS

ABUTMENTS 1 & 2	FRONT STAY LENGTH	REAR STAY	
		LENGTH	PROJECTION
NORTH	3'-8⅜"	4'-6⅚₆"	2'-8⁵⁄₁₆"
MIDDLE	3'-8⅜"	4'-4³⁄₁₆"	2'-6⁵⁄₁₆"
SOUTH	3'-8⅜"	4'-2⁵⁄₁₆"	2'-4⁵⁄₁₆"

Bar 1½" x 3" x 1'-4"
keeper (total 4), typ.

℄ Abut.

Anchor Rod with hardened
washers, lock and jam nuts
(Typ.)

Masonry plate
see detail

** Shear pin

1" ∅ x 2¾" HS bolt
with hardened washer
(Typ.)

* Polytron disc

* Lower bearing plate
a x 1'-4½" x 1'-4½"

2'-2"

2'-2"

2" Typ.

4 spa. @ 3"
= 1'-0" (Typ.)

2" Typ.

℄ Brg.

1'-4½" (Typ.)
Lower brg. plate

2½" Typ.

2½" Typ.

SECTION

TEMPORARY PLYWOOD KEEPER DETAIL
(¾" thick)

2'-2"

4¹¹⁄₁₆"
Typ.

1'-4⅝"

2½" Typ.

℄ Abut.

2'-2"

1'-4⅝"

2½" Typ.

℄ Brg.

1⅝" ∅
hole (Typ.)

MASONRY PLATE DETAIL
(1¼" thick)

2'-2"

4 spa. @ 3"
= 1'-0" (Typ.)

7" Typ.

1⅝" ∅ hole
for anchor rod
(Typ.)

℄ Abut.

¾" Typ.

Threaded hole
for 1" ∅ keeper
bolt (Typ.)

2'-2"

℄ Brg.

2½" Typ.

2½" Typ.

Figure 7 - Disc Bearing Details

DESIGN AND INSTALLATION OF STAYS

Stays need to made from high strength steel to resist the cap beam torsional moments caused by loading one span at a time as girders are set and deck concrete is placed. They can be removed after the deck concrete and abutment (or pier) diaphragms have reached the target design strength, therefore making the girders continuous.

As shown in Figure 6, the lower ends of the back stays were anchored (cast) into the abutment seat while the lower ends of the front stays were secured to the abutment seat with individual anchor rods cast into the abutment seats. The upper ends of both stays were tied to the abutment precast cap with stay plates attached to a common axle as they would be in a typical precast pier cap installation.

Realizing that the 11.5" eccentricity in Figure 6 causes the cap beam to rotate clockwise, the structural behavior of the front stays and the upper ends of the back stays is representative of the behavior in a typical precast pier cap with pinned connections. The behavior of the fixed connections at the lower ends of the back stays was an atypical requirement to accommodate the use of abutment pier caps for cost control reasons as explained in the Demonstration Project Description section.

Stay plates and anchor blocks are required for transferring tension rod forces T to the axles in the supporting members (cap beams and columns) as shown in Figure 8. Eccentricity e creates moment Mx in each stay plate. This moment must be resisted by the stay plates in order to keep the tension rods in pure tension. However, eccentricity e increases as the stay plates are thickened, leading to thicker stay plates to resist a larger Mx.

One way out of this cycle is to size the stay plate thickness to take the bearing load from the axle rod and add stiffener plates to develop the necessary flexural strength in the stay plates to resist Mx. This approach works but increases the cost of the stay plates making it desirable to design the stay plates for reuse on many different projects. Additionally, the axles could be salvaged for reuse on a future project by greasing them to break the concrete bond around them. They could then be jacked out after the superstructure to cap beam connection becomes functional.

10

$$R = \frac{T \cdot e}{j \cdot d}$$

$$M_X = X \cdot R$$

$$j \simeq \tfrac{7}{8}$$

Tension Rod

Anchor Block

Stay Plate Stiffener

Stay Plate

SECTION

Tension Rod

Anchor Block

Stay Plate

Hole for Axle Rod

Stay Plate Stiffener (Typ.)

ELEVATION

Figure 8 - Stay Plate Stiffeners

CAP BEAM LEVER ACTION

Figure 6 shows a typical abutment section wherein the eccentricity of the applied load from the precast girders to the supporting caissons is 11.5" which is the sum of **e1** and **e2** as shown in Figure 9. As per the cap width (B=38") and bearing location that is shown in Figure 6, **e2** is 4" making **e1** 7.5". Using the special equations 1.1 and 1.2 with g=0, Y=11.5", and Z=15" makes T = 0.767P and R = 1.767P.
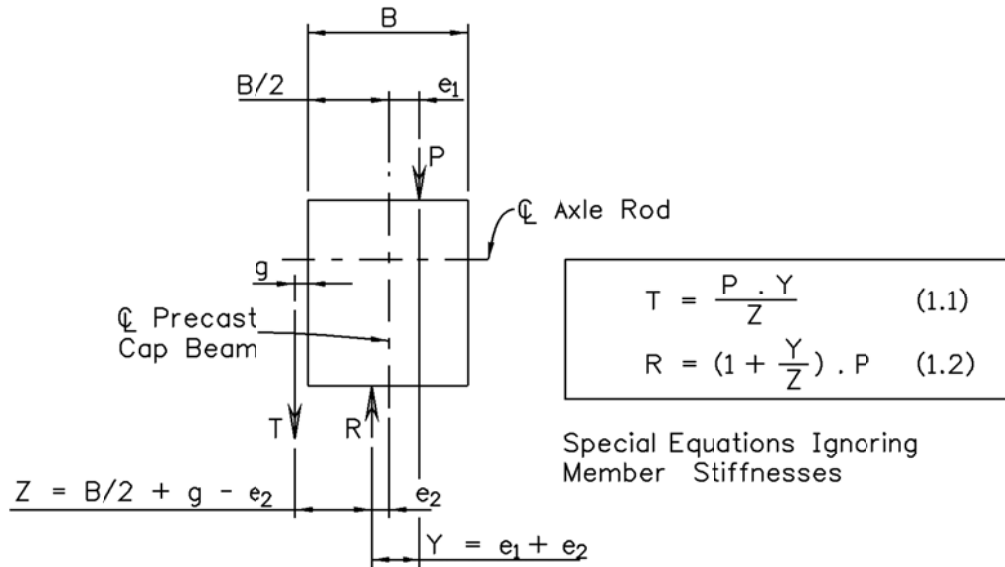

$$T = \frac{P \cdot Y}{Z} \qquad (1.1)$$

$$R = (1 + \frac{Y}{Z}) \cdot P \qquad (1.2)$$

Special Equations Ignoring Member Stiffnesses

Figure 9 - **Special Equations for Abutment Cap Lever Action**

Figure 10 shows a cap beam for a typical pier wherein the centerline of the cap beam lines up with the supporting column. Using the special equations with the same cap width (B=38") and eccentricity (e=11.5"), Y=11.5", Z=19" (for g=0) making T = 0.605P and R = 1.605P. By comparison, the R load for the abutment cap is 10% bigger than the R load for the pier cap and the T load for the abutment cap is 27% bigger than the T load for the pier cap making the abutment cap member an extreme loading for comparison purposes.
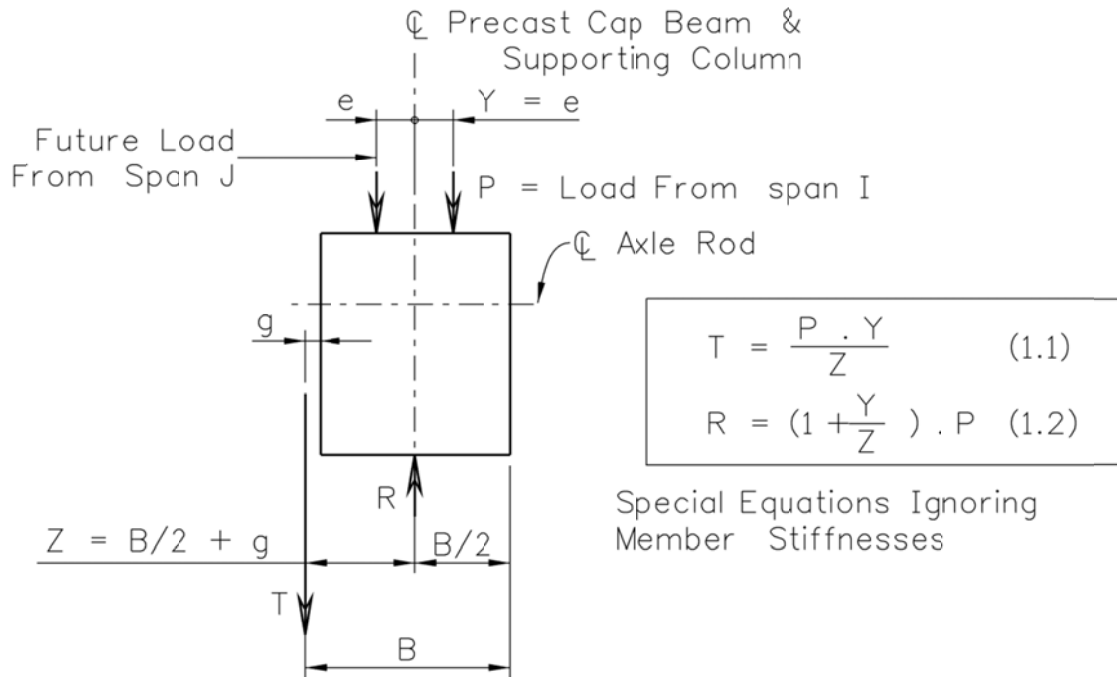
Figure 10 - Special Equations for Pier Cap Lever Action

With respect to the lever action for pier caps, Figure 10 provides the special equations and Figure 11 provides the general equations. The computed stay and bearing forces from the special equations appear to be within 10% of the results from the general equations for normal span configurations. Given the accuracy and simplicity of the special equations, it's appropriate to use them to obtain preliminary member sizes that can be used in the general equations to refine the design during the final design process.
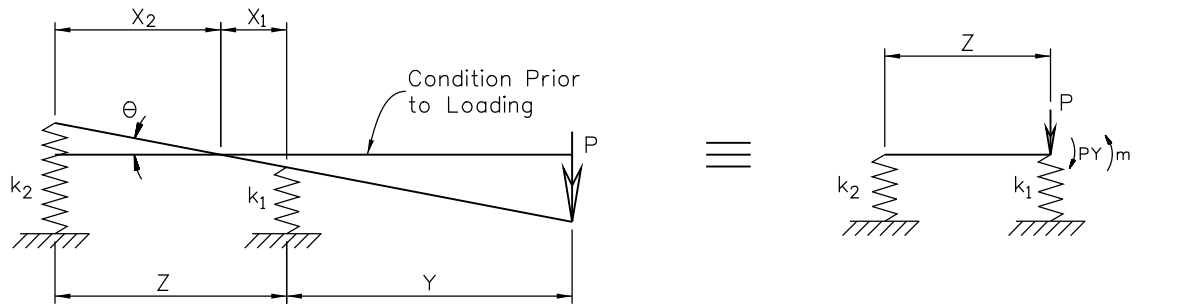
$$\Theta = \frac{\left(Y + \dfrac{Z \cdot k_2}{k_1 + k_2}\right) \cdot P}{\dfrac{\pi \cdot D^4 \cdot E}{64 \cdot t} + \dfrac{k_1 \cdot k_2 \cdot Z^2}{k_1 + k_2}}$$

$$k_i = \frac{A_i \cdot E_i}{L_i} \quad \text{For Axial Stiffness}$$

$$X_2 = \frac{k_1 \cdot Z \cdot \Theta - P}{(k_1 + k_2) \cdot \Theta}$$

k_1 = Bearing Axial Stiffness

k_2 = Stay Axial Stiffess

$\Delta_2 = X_2 \cdot \Theta$ = Stay Tension Deflection

$$\frac{m}{\Theta} = \frac{\pi \cdot D^4 \cdot E}{64 \cdot t} = \text{Bearing Rotational Stiffness}$$

$X_1 = Z - X_2$

$\Delta_1 = X_1 \cdot \Theta$ = Bearing Compressive Deflection

$T = k_2 \cdot X_2 \cdot \Theta$ = Stay Tension Force

$R = k_1 \cdot X_1 \cdot \Theta$ = Bearing Compression Force

$$m = \frac{\pi \cdot D^4 \cdot E \cdot \Theta}{64 \cdot t} = \text{Bearing Moment}$$

Figure 11 - General Equations for Pier Cap Lever Action

The equations in Figure 11 allow the net area for the disc bearing to be used in the axial stiffness computations and ignore the presence of a shear pin (pintle) in the rotational stiffness terms. The predicted rotations using this design methodology were about half of the observed rotations during construction as explained in the Cap Beam Rotations section below.

AXLE BEARING STRESSES

As shown in the Section in the installation details in Figure 12, a #6 spiral rebar and a weld plate are part of the axle assembly. The spiral is required to provide confinement to increase the concrete bearing capacity around the axle and the combination of the axle and the spiral form a strong-back. The weld plate is required to (1) prevent moment from developing in the rear stays caused by positioning errors during construction, and to (2) serve as a side plate to reduce the load on the strong-back via the shear studs. Hence, the need for a weld plate, as explained below, is limited to the abutment installations on this bridge while the need for side plates and spirals applies to all precast pier caps.

The ultimate compressive strength for confined concrete can been estimated with the following equation[1]:

$$\mathbf{\textit{f'cc} = \textit{f'c} + 4.1\textit{fcon}} \qquad (1.3)$$

The term **fcon** in this equation is the hoop-tension confining pressure that can be provided by the spiral rebar. The observed maximum value in the reference document for **f'cc** is 80 ksi.

The LRFD bridge design code[2] provides the following equations for the factored bearing resistance:

$$\mathbf{\textit{Pr} = \textit{Ø*Pn}} \qquad (5.7.5\text{-}1)$$

> where Ø is a resistance factor with a value of 0.70.

$$\mathbf{\textit{Pn} = 0.85\textit{f'c*A1*m}} \qquad (5.7.5\text{-}2)$$

> where **A1** is the projected area of the axle and **m** is a modification factor to recognize the benefits from confinement with an upper limit of 2.0.

For the purpose of calculating the axle bearing stresses, **f'c** was replaced with **f'cc** in equation 5.7.5-2:

$$\mathbf{\textit{Pn} = 0.85\textit{f'cc*A1*m}} \qquad (1.4)$$

For **f'c** = 4500 psi, the #6 spiral rebar with an 8" outer diameter and a 2.25" pitch creates **f'cc** = 15.32 ksi inside the spiral which is less than the 80 ksi maximum that was observed by Collins and Mitchell[1]. Using rigid-body mechanics, this increase in strength alone is not enough to provide the required support strength along the axle without

---

[1] "Prestressed Concrete Structures", Collins & Mitchell, 1997, Response Publications, Canada, equation 3-22 on page 79

[2] AASHTO LRFD Bridge Design Specifications, fifth edition, 2010

setting m = 4.0. Accordingly, a side plate with shear studs must be used to reduce the axle shear force to an acceptable level that can be resisted with m = 2.0.

PRE-CAST CONCRETE CAP TEMPORARY INSTALLATION NOTES

1. Individual Bearing Reactions:
   a. See Precast Cap Beam Self Weight and Deck Dead Load Reaction Table for the initial load at each bearing caused by the weight of the precast cap beam and superstructure dead load.
   b. Service reaction = 435K DL + 60K LL = 495K total each bearing during construction.
   c. Strength reaction = 544K DL + 90K LL = 634K total each bearing during construction.
   d. Service reaction = 354K DL + 167K LL+IM = 521K total each bearing after construction.
   e. Strength reaction = 450K DL + 293K LL+IM = 743K total each bearing after construction.

2. Erection Sequence:
   a. Install shim stacks to proper elevations on abutment seats and install temporary plywood keepers to maintain position when cap is placed. Install Type III Bearings and tighten masonry plate nuts. Determine need for shims at weld plates in precast cap and have them fabricated and delivered to the site prior to erection.
   b. Place cap beam in plumb position on bearings using crane to prevent cap beam rotation.
   c. Install and snug tighten the temporary stays at each bearing without bending or kinking all-thread tie rods. Weld slotted hole plate to shim plate and shim plate to weld plate as called for in the section. Weld upper bearing plates to precast cap embedded plates. Loosen stab end nut ½ turn at seat stay plate.
   d. Release the crane and use the bearing load measuring rings to record the cap dead load reaction at each bearing. Install temporary hard wood safety wedges as shown on sheets B7 and B8.
   e. Erect the precast girders.
   f. Grout the vertical bars in the ends of each girder. Tamp plug into fresh non shrink grout and weld as called for in the section one day later.
   g. Remove the temporary plywood keepers and solid grout the space underneath the Type III Bearing masonry plates with non-shrink grout.
   h. Install backwall and remove formwork after it has cured for three days.
   i. Place the diaphragm and deck concrete. Use the bearing load measuring rings to record the superstructure reaction at each bearing immediately after placing deck concrete.
   j. After deck concrete has reached full strength, remove exposed stays and flame cut the all-thread rods in the rear stays next to the backwall. Remove temporary hard wood safety wedges.
   k. Cut the exposed stay anchor bolts off 1" below the surface of the finished concrete and fill and finish with an approved epoxy grout.

3. Materials:
   a. Threaded and smooth rods, AASHTO M-314 (ASTM F1554) Grade 105.
   b. High strength nuts, AASHTO M-291 (ASTM A563).
   c. Hardened washers, AASHTO M-292 (ASTM A436).
   d. Plates AASHTO M-270 (ASTM A709) Grade 50.
   e. Bars AASHTO M-270 (ASTM A709) Grade 50.
   f. Hard wood blocks, NELMA No. 1 Red Oak with compression perpendicular to grain = 820 psi min.
   g. E70 electrodes.

PRECAST CAP BEAM SELF WEIGHT AND DECK DEAD LOAD REACTION TABLE (SERVICE LOADS)

BEARING	ABUTMENT 1 (K)		ABUTMENT 2 (K)	
	† SELF WT.	* DECK DL	† SELF WT.	* DECK DL
North	27.96	428.09	27.99	428.12
Middle	29.98	428.15	30.61	428.78
South	27.85	428.00	32.90	433.05

† See note 2d
* See note 2i

SHIM TABLE

Shim (t) thickness	x	y
0	⅜	6
⅛	½	14
¼	³⁄₁₆	12
⅜	⁵⁄₁₆	7

SHIM DETAIL
(Install shims as needed when embedded tie rods are not in a common vertical plan – see note 2a)

ANCHOR BLOCK DETAIL
(Bar 4" X 4")

ROUND HOLE
(at seat)

ROUND HOLE
(at precast cap)

STAY PLATES
(1½" Thick)

SLOTTED HOLE
(at precast cap)

SECTION

WELD PLATE DETAIL
(¾" thick)

✱ See table on sheet B11a for tie rod lengths

Figure 12 - Installation Details

17

As per the load path, the load goes from the axle to the side plate and spiral, and from the side plate and spiral to the surrounding cap beam concrete. As previously explained, the shear stud capacities in the side plates are required to reduce the bearing stresses on the spiral to acceptable levels. Rigid-body mechanics can be used to check the axle bearing stress with a side plate. Confinement from #5 stirrups as shown in Figure 13 produce an effective concrete stress with ***f'cc*** =6.91 ksi for the strong-back which in turn limits the required axle bearing stress inside the spiral to a manageable value computed with ***m*** = 2.0.

Figure 13 - Cap Beam Details

19

The need for side plates prompts the need for a square column cap as shown Figure 2 and as detailed in Figure 14 in order to avoid complications in the fabrication of the side plates. Additionally, the square shape facilitates the installation of the confinement and bursting steel that is identified in Figure 14. The column cap could be a cube if D minus 2" for clearance is greater than, or equal to, the required development length for the longitudinal reinforcement in the column.
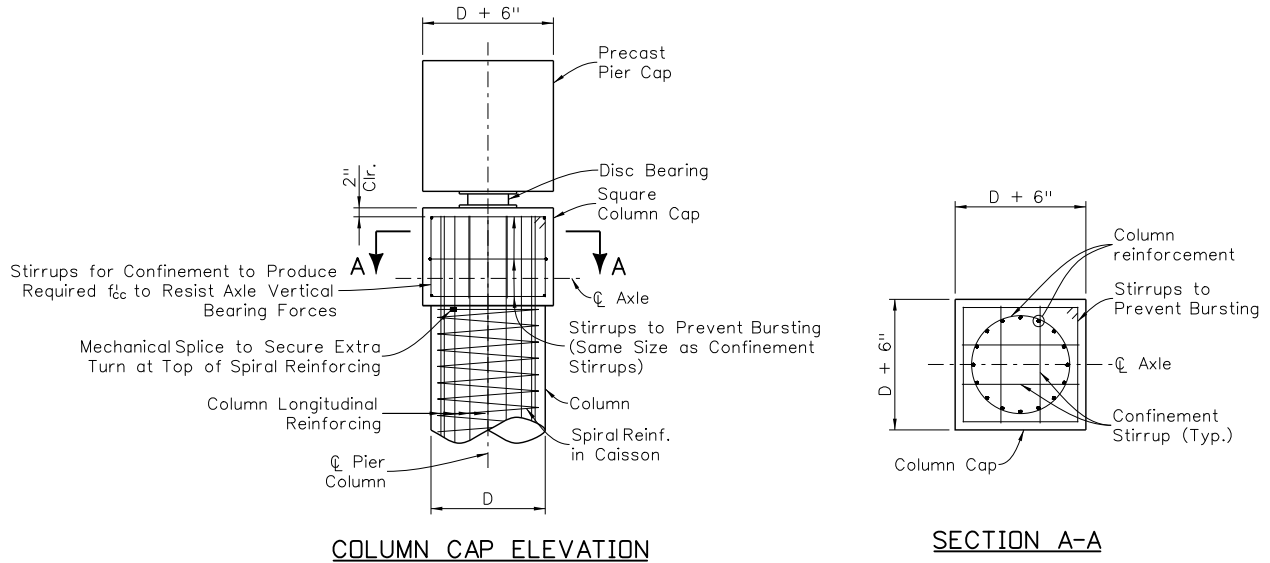
Figure 14 - Square Column Cap Details

ERECTION TIME

It took two hours to set three bearings at the west abutment and just one hour to set three bearings at the east abutment after everyone became familiar with the process (See Figure 15). Likewise, it took four hours and twenty minutes to set the cap beam at the west abutment and just two hours to set the cap beam at the east abutment (See Figure 16).

Figure 15 – Setting of Bearings
(Note: A special clamping bar was used to lift the bearings.)

Figure 16 – Setting of cap beam
(Note: The crane supported some of the cap's weight while
the stays were connected and hand tightened.)

All of the stays went into their stay plates easily. However, to assist in the process, grinding a ¼" bevel on the top ends of the all-thread tie rods and stays would help to eliminate thread damage during the "stabbing" operation (the insertion of the stays into the stay-plate anchor-blocks as the cap was lowered into position). This, in turn, makes it easier to turn the nuts onto the rods and stays by hand.

After the caps were installed, it took six hours to install seven girders using two cranes. Hence, the total erection time for installing six bearings, two caps, and seven girders was about 15 hours. Setting the girders required two cranes and some interesting teamwork (See Appendix A).

CAP BEAM ERECTION PROCESS

Disc bearings are very rigid, having a relatively large compression area with a compressive modulus of 55 ksi. Consequently, they won't compress much from the cap beam weight; on this project the upper bound was 0.01".

The practical vertical tolerance for the placement of bearings was found to be 1/16", making the maximum relative difference in elevation between adjacent bearings 1/8". Consequently, the cap beam would not be supported by the lowest of the three bearings. Additionally, our calculations showed that the cap beams wouldn't deflect more than 1/32" for a 29'-10" simple span even if it was supported on only the outer two bearings as shown in Figure 17.

Accordingly, a supply of 1/16" and 1/32" thick shims was needed during erection to raise the height of the lowest bearing. The shims were used to eliminate unwanted gaps between the upper bearing plate and the embedment plate as shown in the Typical Section on Figure 7. The width and length of these shims must be the same as the width and length of the lower bearing plate in order to fit between the anchor rods.

About 25% of the cap beam weight must be held by the crane during erection to determine the shimming requirements at the low bearing. The cap beam must be raised about an inch to facilitate placement of the required shims. It is then lowered, plumbed and the stays tightened. With the cap beam plumb and the stays tightened, the crane can be released to accommodate the field welding process as called for in Figure 7.

PLAN

ELEVATION

END VIEW
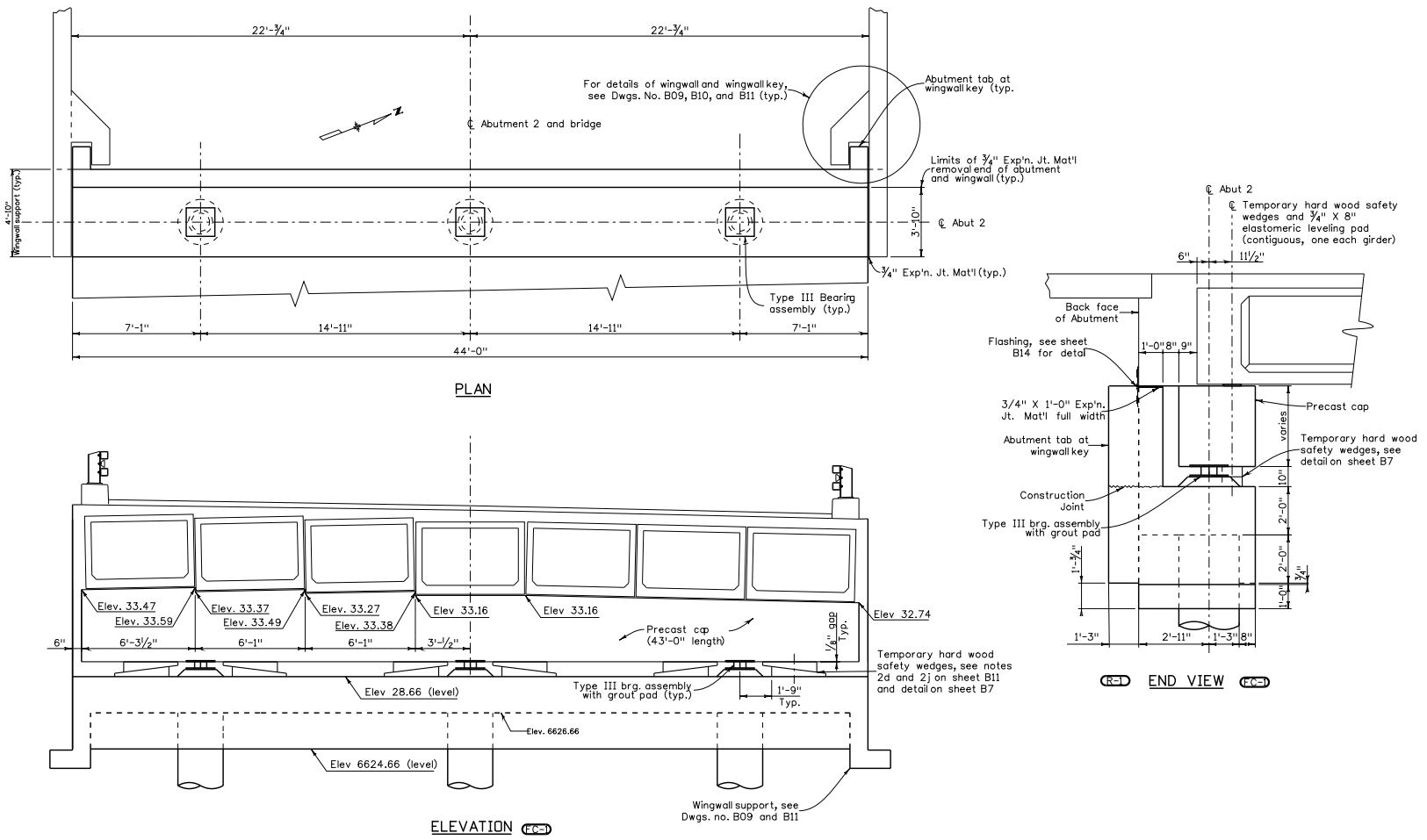
Figure 17 - Abutment Details

24

INDIVIDUAL BEARING REACTIONS

RJ Watson disc bearings were selected for use on this project because they could be fitted with proprietary Load Measuring Rings (LMRs) to accurately measure the bearing loads at each stage of construction. For quality control reasons, a bore gauge was used to verify the accuracy of the LMRs.

Unfortunately, the bore gauge readings did not match the LMR readings and neither method yielded the predicted loads. It was concluded that the construction tolerances as described in the Cap Beam Erection Process could create a partial contact scenario requiring a "differential settlement analysis" to verify the observed readings. However, this knowledge alone was not comforting because the cap beam rotated noticeably during construction introducing a second source of error for the LMR readings (the LMRs were calibrated in a laboratory using parallel loading surfaces).

With all of this in mind, the bore gauge readings seemed more appropriate because the readings could account for the cap beam rotations. This was done by taking readings on the bending neutral axis for the bearings which is parallel to the cap beam longitudinal axis. Additionally the LMR readings were changing from week to week indicating that loads were being redistributed from the higher stressed bearings to the lower stressed bearings.

It was concluded that construction tolerances could lead to the observed bore gauge readings. The disc bearings were found to be robust and capable of redistributing uneven loads via plasticity, without incurring any permanent damage, as long as the loading surfaces were within 1/32" of where they should be. This was achievable as described in the Cap Beam Erection Process.

CAP BEAM ROTATIONS

With respect to the unsymmetrical loading conditions shown in Figure 6, the cap beam rotated clockwise about 1/8" per foot at each abutment as the girders were placed. Figure 11 provides the general equations for predicting the stay forces, bearing forces, and cap beam rotations during construction. Using a compression modulus for the disc bearings from the manufacturer of 55 ksi gave us cap beam rotations that were roughly half of the observed rotations.

The partial contact problem as discussed in the section on Individual Bearing Reactions may be the source of this error. Accordingly, the theoretical rotations should be doubled to approximate what will actually occur during construction.

For esthetic reasons, hydraulic jacks were used to rotate the cap beams backwards on this bridge to compensate for the anticipated rotations due to the placement of a CIP deck. It would not be possible to make this adjustment at a typical pier cap installation because there is no room on the columns to set the jacks.

From a structural viewpoint, there is no reason to be concerned about cap beam rotations as long as the rotations don't exceed the bearing rotation limits. Accordingly, it is important to check to see if the bearing rotation limits will be exceeded during construction using twice the predicted rotations from the placement of girders in one span only.

As shown in Figure 11, the theoretical equation for rotations assumes that the rotation will take place on the centerline of each bearing that is parallel to the centerline of the cap beam. As per the rotation calculations for the cap beam plus girders and for the cap beam plus girders and CIP deck, this assumption was valid and probably will be valid for any pier cap rotation calculations as long as the bent skew angle is not extreme.

LESSONS LEARNED

Even though the bearings on this project were not damaged by the observed uneven loading conditions, it might be best to anticipate and design for a reasonable worst case loading condition as shown in Figure 18. In this scenario, the smallest loads from the lightly loaded bearings and the maximum load from the overloaded bearings could be used to design a typical bearing for use in the bent of concern.
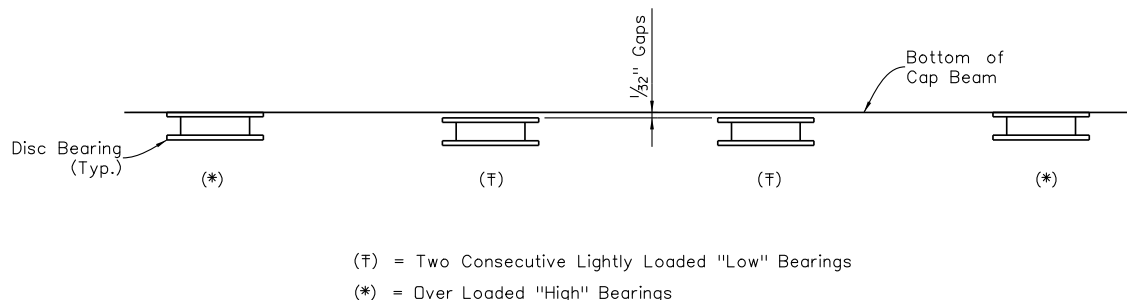

Figure 18 - **A Reasonable Worst Case Loading Scenario**

Long term creep effects were observed. Recorded LMR readings show that long term creep effects are measured in weeks as opposed to hours and days. Additionally, Section 14.7.8.3 in the AASHTO LRFD code implies that properly manufactured urethane discs may have permanent creep deflections equal to 8% of their original thickness. With the 1/32" gap as shown in Figure 18, the 8% creep factor leads to a minimum bearing height of 0.39" which is less than the 1.25" dimension that is specified in Figure 7 implying that there would be no reason to consider overloaded and lightly loaded bearings in the design phase. This logic inadvertently supports the design process that was used on this research project.

As a point of reference, pot bearings and laminated elastomeric bearings do not have significant long term creep effects. The elastomer in a pot bearing is confined making it impossible for this material to deflect vertically because it's incompressible. The thin layers of elastomer in a laminated bearing probably do creep some, but not significantly. Accordingly, the use of these two bearing alternatives would require precise shimming to prevent the creation of permanently overloaded and lightly loaded bearings during and after construction. Hence, the long term creep effect in disc bearings may be an advantage for ABC using a mix of precast and CIP elements.

For the cap beam in a bent supporting unsymmetrical spans, the guidelines for estimating rotations in the Cap Beam Rotation section above should be sufficient for determining an initial rotation angle to end up with a level cap after the girders and deck in both spans have been placed.

Easy access to the bearings in a bent is needed during the erection of a pier cap to facilitate (1) the selection and placement of leveling shims between the upper bearing plate and the embedded plate at each bearing as shown in Figure 7 and (2) welding of the upper bearing plates to the embedded plates as called for in Figure 7. With erection time in mind, it might be worthwhile to install scaffolding along each side of a bent from end of cap to end of cap while the columns are curing. Several man-lifts will be required if scaffolding is not an option.

APPENDIX A – PROJECT PHOTOGRAPHS

Structure N-16-L at mm 18.9 on SH-69 northwest of Walsenburg was built in 1923.

Girders for the replacement structure, N-17-Q arrived at the site on trucks. The span was too wide for a single crane to set the girders so a pair of cranes, one at each end of the structure, positioned the girders. The east end of a girder was lifted from its truck by the crane at the west abutment.

The crane at the west abutment swung the east end of the girder out over Turkey Creek to the point where the crane at the east abutment could assume the load by hooking the triangular lifting device as shown in the following pictures.

The eastern crane hooked the open shackle on the triangular lifting device.

While the western crane held the east end of the girder, the eastern crane hooked the open shackle on the lifting device and slowly lifted it to assume the load.

The eastern crane began to assume the load.

Once the open shackle was hooked, the eastern crane slowly raised its hook to take the load. The western crane slowly lowered its hook to transfer the reaction at the east end of the girder to the eastern crane.

Once the eastern crane assumed the reaction at the east end of the girder, the western crane slowly lowered its hook and disconnected from the triangular lifting device.

All of the load was on the eastern crane.

With the eastern crane supporting the east end of the girder, the western crane swung back and lifted the west end of the girder from its truck and the two cranes set the girder in position.

Both cranes begin to lower the girder onto the abutments.

APPENDIX B – FIELD NOTES AND DATA

Field notes for IBRD (Innovative Bridge Research and Development Program)
Structure N-16-Q on SH-69 at Turkey Creek

This section contains notes and instrument measurements taken during the placement of the caps on the abutments. The notes were made by a CDOT Research Branch observer who is not a bridge engineer. They include observations on the installation with comments about some of the steps and some suggestions that might make future installation of similar structures more efficient and faster.

10/13/10 – The contractor began installing bearings on the west abutment at about 0830. The west abutment was completed about 1030. Several minor things slowed the installation of the first bearings:

- The grout used below the bearings was mixed too dry at first and was placed on a dry abutment surface. The grout needs to be placed on a wet surface and the mix itself must be fluid enough to allow the weight of the shim stack to spread it uniformly across the bearing surface.

- The openings in the temporary plywood keepers for centering the shim stacks didn't have enough tolerance. The lack of proper tolerance made it difficult to center the shim stacks after grout was placed. The openings needed to be large enough to allow the steel shim stacks to be centered but small enough to hold them in their appropriate positions. If the keeper openings had been cut ¼ inch oversize in both directions, the shim stacks would have been easy to place and center in the openings.

- The device used to lift the bearings into position must be designed to hold the bearing securely as it is positioned and to clear the anchor bolts and release the bearing easily. The original design did not clear the anchor bolts and had to be modified to allow the bearings to be set in place.

- During the bearing placement lateral movement damaged the ends of the anchor bolt threads which made installing the nuts difficult. A ¼ inch chamfer on the ends of the bolts would prevent most of the damage. Having a suitable sized thread-chasing tool would make it easy to clean up damaged threads so the nuts could be started on the bolts by hand without the use of a wrench. A ratchet wrench with a deep socket would greatly facilitate the installation and tightening of the top nuts.

Installation of the three bearings on the east abutment took about one hour, and went much more smoothly because of the lessons learned on the west abutment.

The bearings used on the project were provided by R. J. Watson, Inc. They have load measuring rings to allow measurement of the load being supported by each individual bearing. A cable, anchored at one end, wrapped around the polytron bearing, and connected to a potentiometer converts the increase in the bearing's diameter to a resistance reading that can be measured at a jack on the side of the bearing. The temperature of the bearing plates above and below the polytron disk bearing are a factor in the expansion of the bearing. Their temperatures were measured at various stages during construction using a Heat Spy® infrared thermometer.

We took resistance and temperature readings on the bearings after they were placed on the abutments. They are the "Install" column in the table below. Even though the

temperatures of the bearings changed significantly, the pre-cap readings taken on the 20th did not change much from those taken when the bearings were installed on the 13th. Since both sets of readings were with no load, no change was expected.

10/20/10 –Installed both caps.

0800 – The crane (Gary's Crane Service of Pueblo – 175T) was on site and setting up. He started to make the first lift, then stopped and reconfigured his block from 6-wire to 8-wire. (I believe it was so he would have better fine control when setting the caps.)

1025 – Lifted the west cap from its truck and swung it into position. All of the stays went into their stay plates easily. The back stay at L6 – the north bearing on the west abutment – was too long (The stay was longer than called for in the drawings.) and interfered with the axel nut. The top of the stay had to be cut off about 3 inches.

1115 – All of the stay bolts have been installed. Erection Sequence #b (sheet B11 in the plans and **Error! Reference source not found.** in this report) calls for the cap to be placed in plumb position. The contractor asked which face of the cap was to be plumbed. My recommendation was to use the bottom of the cap since it was the reference plane used when the cap was poured at Rocky Mountain Prestress.

1200 – Welding the stays and bearing plates started at L6. At L5 the cap did not sit on the bearing plate. The gap was about 1/8 inch. Tightening all stays and releasing all weight from the crane reduced the gap to about 1/16 inch – slightly less on the north side of the bearing. Staff Bridge was consulted. The decision was to go ahead with welding the bearings and stay plates.

1445 – Welding completed on the west cap. The crane and equipment moved to the east side of the bridge.

1545 – The east cap was lifted and positioned. The ends of the L3 and L6 back stays had to be cut in order to miss the axle assemblies.

1645 – Welding began at L3

1745 – Welding completed

Winslow's crane arrived about noon and began setting up and attaching counterweights. Girders were scheduled to arrive on the 21st at about 0700.

10/21/10

0800 - The girders arrived and the first three were set to the middle of the caps. Gary's crane could not reach farther than the middle of the cap with the weight of the girder.

1030 – Repositioned Gary's crane to the middle of the east cap.

1145 – The first three girders were placed in their final locations and Gary's crane moved back to the north end of the east abutment.

1400 - The girders are in their final locations. Both cranes began break-down and I took temperature and gage readings and pictures of the bearings. As the table below shows L1, L2, and L4 readings looked good. L3 and L6 broke and L5 is questionable.

Table 1. Instrument readings and temperatures during the installation of the bearings, caps, and girders.

Brng #	Bearing Position	10/13/10 Brg. Install (°F/kΩ)	10/20/10 No load (°F/kΩ)	10/20/10 Cap placed (°F/kΩ)	10/21/10 Girders placed (°F/kΩ)	Deck (°F/kΩ)	Comments
L1	E Abutment south	51.5° – 54.5°/2.28	87.7° – 80.6°/2.27	87.8° – 79.5°/2.27	53.5° – 51.5°/1.87		Gage Δ = 0.40 kΩ
L2	E Abutment middle	66.3° – 60.1°/2.13	85.8° – 75.1°/2.13	81.9° – 79.0°/2.12	54.8° – 53.8°/1.91		Gage Δ = 0.22 kΩ
L3	E Abutment north	63.4° – 63.8°/2.38	84.3° – 79.2°/2.40	79.8° – 77.4°/2.39	NA		Cable broke at the fixed base post*.
L4	W Abutment south	55.0° – 52.0°/2.27	44.5° – 42.7°/2.28	71.6° – 67.4°/2.21	53.3° – 52.2°/1.78		Gage Δ = 0.49 kΩ
L5	W Abutment middle	49.0° – 52.0°/2.50	43.6° – 45.8°/2.52	78.2° – 70.0°/2.50	53.3° – 52.4°/2.31		Gage Δ = 0.19 kΩ Very small change – possibly due to gap between the bearing and cap.
L6	W Abutment north	58.2° – 54.6°/2.41	44.7° – 42.5°/2.39	71.4° – 67.1°/2.38	53.3° – 52.1°/2.37		Gage Δ = 0.04 kΩ The spring tube* broke, apparently jamming the potentiometer*.

Notes: "°F" is upper- and lower-bearing plate temperature measured with the Heat Spy close to the gage electronic jack*.
The top temperature in each cell is the upper plate*, the bottom is the lower plate* - the number after the slash is the resistance reading.
* R. J. Watson, Inc. names for parts of their load measuring device (LMD).

Things that might help facilitate future installations:

- If all threaded stays, rods, and axles were chamfered ¼" on the end, damage to the threads during the handling of the parts would be reduced. An alternative (possibly less expensive and more certain) would be to require a universal thread chasing tool to be present on the job site. Damaged threads make it much more likely that the nuts will be started crooked which will damage the threads further.

- The grout used below the steel shim stack should be mixed "runny" and be applied to a wet surface on the abutment.

- It is much easier to apply the grout before the plywood keeper is positioned.

- The cut-outs in the temporary plywood keepers used to position the shim stacks need to be large enough (perhaps ¼" tolerance in each direction) to allow the shim stacks to be centered after grout has been placed.

- The device used to lift and position the bearings must clear the anchor bolts and be easy to remove from the bearings after they are set on the bolts. It also needs to keep the bearing very close to level as it is lowered into position. If it tilts even slightly, the holes in the masonry plate will jam on the anchor rods and damage the threads.

- A socket wrench and ratchet handle would make installing the lock nuts (the top nut on the anchor bolts) much easier and faster than using an open end wrench.

- The erection sequence needs to specify which surface of the cap is to be set plumb.

- Several 1/32", 1/16", and 1/8" thick steel shims should be available and sized to fit the top of the bearings without interfering with the weld to the embedment plates. During the cap placement, they can be placed on top of low bearings that do not support the cap.

- A pre-install inspection procedure should be developed or requested from the bearing manufacturer – R. J. Watson, Inc., in this case. A pre-install inspection might have located the problems with the instrumentation cable and spring tube assembly before the bearings were installed and the caps set. It might have been possible to fix them before installation.