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Bridge Construction Records And Procedures

Department Of Transportation
Structure Construction

Volume 2

Updated August 31, 2015

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STEVE ALTMAN
Deputy Division Chief
Structure Construction
Division of Engineering Services

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4-1.07 Value Engineering

4-1.07A General

Construction Manual
Additional information is available in the *Construction Manual*, Section 3-405, *Value Engineering.*

4-1.07B Value Engineering Change Proposal

Resources, Review and Determination Procedure
Additional information is available in the:

- *Structure Design Alert*[1] dated July 1, 2014.
- Project Delivery Directive, PD-13, *Value Engineering Change Proposals*.[2]

The Division of Engineering Services (DES) is responsible for providing technical concurrence on structure related Value Engineering Change Proposals (VECPs). Acceptance of the VECP is a district/region responsibility. The Deputy District Director for Construction (DDDC) in each district makes the final determination if a VECP is acceptable based in part on the technical recommendation of DES.

It is critical that VECPs are thoroughly vetted and not rejected or accepted without full consideration. If a determination is made that a structure related VECP is:

- Acceptable to DES, that decision should be made at the lowest level possible and concurred at the level just above those making the decision.
- Unacceptable to DES, that decision must be validated by an appropriate team of DES Deputies (Structure Construction [SC], Structure Design [SD], and Material Engineering and Testing Services/Geotechnical Services [METS/GS]) before the determination is transmitted to the district.

Proposal Concept Stage
When the proposal concept is presented, the appropriate DES Representatives (Structure Representative, Structure Design Project Engineer, Bridge Construction Engineer (BCE), and SD Branch Chief) should participate in the meeting between the Contractor and the Engineer. To ensure an adequate understanding of the *Proposal Concept*, invite all parties from DES[3] and Structure Maintenance & Investigations, to participate as appropriate. It is important that fatal flaws in the *Proposal Concept* be discussed early before the Contractor prepares a VECP.

The DES Representatives must **confer and concur as to the DES decision for the *Proposal Concept***. If a *Proposal Concept* is found to be unacceptable (rejected), the next level (Area Construction Manager (ACM) and SD Office Chief) should review it. If they concur with the

[1] http://onramp.dot.ca.gov/hq/des/sd/docs/structure_design_alerts/sda_20140701.pdf
[2] http://www.dot.ca.gov/hq/projdev/directive/PD-13-VECP-3-14-14.pdf
[3] Appropriate DES parties could include Geotechnical Services, Earthquake Engineering, Structures Office Engineer (Specifications), Hydraulics, etc.

rejection, the *Proposal Concept* and VECP Analysis Report should be forwarded to the next level (DES Deputies) for review. **The DES Deputy Division Chiefs for SC, SD, and METS/GS will make the determination for DES on whether the VECP *Proposal Concept* is unacceptable**.

Submit the DES decision in writing to the Resident Engineer. The decision to proceed with the *Proposal Concept* should be made at the lowest level possible and concurred at the level just above those making the decision. **The decision to accept a *Proposal Concept* must be documented on the *VECP Analysis Report***, for which a template is provided in Attachment No. 1.

For structure related items, the review times required by DES will vary depending upon the complexity of the *Proposal Concept*, as multiple functional units within DES will need to review and provide input on the VECP. Convey to the Resident Engineer the review times required by DES to the Resident Engineer for their discussion with the Contractor.

VECP Investigation Stage

During the VECP investigation stage, the BCE will facilitate the review by all DES functional units[4] and Structure Maintenance and Investigations to ensure all stakeholders have provided input and to ensure timely completion of the investigation.

The DES functional unit representatives should confer and concur as to the DES decision for the VECP. If the representatives find the VECP to be unacceptable, the next level (Area Construction Manager (ACM)/Office Chief) should review the VECP; if they concur with the rejection, the BCE should arrange for a meeting between the DES Deputy Division Chiefs for SD, SC, and METS/GS to review the VECP. **Prior to the meeting, provide the *VECP Analysis Report* to the Deputy Division Chiefs.**

The *VECP Analysis Report* will provide:

- All pertinent contract information along with a description of the VECP.
- Structures affected.
- Positive aspects of the VECP.
- Reasons the VECP should be rejected.
- The recommendation(s) of the Structure Representative and Project Engineer.

Submit the DES decision, in writing, to the Resident Engineer. The decision to proceed with the VECP should be made at the lowest level possible and concurred at the level just above those making the decision. **The decision must be documented in the *VECP Analysis Report*.**

Roles / Responsibility

Structure Representative – Collaborate with the Structure Design Project Engineer on the DES recommendation to the VECP.

[4] SD, METS/GS, and Program/Project and Resource Management (PPRM)/Special Funded Projects

Structure Design Project Engineer – Evaluate the VECP for technical soundness and collaborate with the Structure Representative on the DES recommendation to the VECP.

Bridge Construction Engineer – Facilitate the review by all DES units to ensure timely completion. If needed, arrange for a meeting with the Deputy Division Chiefs from SC, SD, METS/GS and PPRM. Work with the ACM to review the Structure Representative's and Project Engineer's recommendation to the VECP.

Structure Design Branch Chief – With the BCE, review the Structure Representative's and Project Engineer's recommendation to the VECP.

Area Construction Manager – In cases of rejection of the VECP, verify that the VECP has been properly vetted. Confer with the SD Office Chief on the recommendations to the VECP. Meet with the Deputy Division Chiefs to review the cause for rejection. Confirm the recommendation from DES to the District, or return the VECP to the Structure Representative and Project Engineer for continued review.

Structure Design Office Chief – In cases of rejection to the VECP, verify that the VECP has been properly vetted. Confer with the ACM on the recommendations to the VECP. Meet with the Deputy Division Chiefs to review the cause for rejection. Confirm the recommendation to the District or return the VECP to the Structure Representative and Project Engineer for continued review and evaluation.

Division of Engineering Services Deputy Division Chief – In cases of rejection of the VECP, verify that the VECP has been properly vetted. Confirm the DES recommendation to the District or return the VECP to the Structure Representative and SD Project Engineer for continued review and evaluation.

The following is a sample *VECP Analysis Report* that can be used as a template:

Structure Construction – Value Engineering Change Proposal Analysis (VECP) Report

Insert Date

Project Information

Dist-EA
Dist-Co-Rte-PM
Structure or bridge name
Br. No.

Description of VECP

Provide a description of the VECP

- *Reduce any cost of construction.*
- *Reduce construction activity duration.*
- *Reduce traffic congestion.*
- *Permit issues.*
- *Impact on other projects.*
- *Project impacts, including traffic, schedule, later stages.*
- *Peer reviews.*
- *Overall proposal merits.*
- *Review times required by the Department and other agencies.*
- *Etc.*

Structure(s) Affected: *(Identify any structures that are affected)*

Chronology:

Proposal Concept received: *(date)*
Proposal Concept accepted or rejected: *(date)*
VECP received: *(date)*
VECP accepted or rejected: *(date)*
Change Order issued: *(date)*
Elapsed review time: _____ days

Introduction:

This report presents the results of the review for the *(insert type of review completed, i.e. Proposal Concept or VECP.*

Discussion:

<u>Positive Aspects of the VECP</u> – *List and clarify*

Structure Construction *Value Engineering Change Proposal (VECP) Report* continued

Reasons the VECP should be rejected – *This portion of the report would describe specific deficiencies found with the Proposal Concept or VECP that would be cause for rejection i.e.*

Recommendation of the Structure Representative:
Authorization – No exceptions were found with the VECP (*number or title of VECP or other unique identifier).*

Rejection:
Structure Construction does not accept the VECP. The *(insert type of review completed, i.e. Proposal Concept or VECP)* for *(identify specific location)* of the (*Bridge name, Br. No.),* based upon the analysis that found the deficiencies listed above.

If you have any questions regarding this report, please contact Structure Representative at (XXX) XXX-XXXX.

Steve Street, P.E.
Structure Representative
Structure Construction

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51-1.03C(2) Forms

51-1.03C(2)(a) General

Industry Practices

For other than exposed surfaces form panels may consist of surfaced lumber, plywood, steel and in some cases synthetic materials, depending on the type of construction and the surface finish required. For exposed surfaces the degree of care taken by the contractor when building forms often determines the amount of subsequent work needed to obtain the required finished surface. Below are some industry acceptable practices:

1. Form panels that have minor damage or are damaged (damaged corners, holes, delaminations, and scars) after installation, and cannot be reasonably replaced, may be repaired when using acceptable method and materials, such as wood fillers, resin products (Bondo), and wood or cork plugs.
2. Expanding foam may be used to ensure forms are placed flush against concrete in areas around columns and abutments. Care must be taken when determining the product's expansion capabilities. Excess foam that protrudes into the concrete section must be trimmed off.
3. Acceptable materials to form the exterior girder and soffit radius sweep may include steel, PVC, or 1/8″ veneer plywood sheeting. All materials must be adequately supported at the joist and throughout the spans to prevent the development of discontinuities between form panel sections during concrete placement.
4. New plywood next to old plywood will produce an unacceptable non-uniform concrete surface. One method proven successful to age the new forming material is to apply cement and water paste, allow the paste to dry, and then remove it. The dried cement paste absorbs the fresh wood sugar from the new plywood and ages the wood so that the finished concrete will have a color and texture similar to the color and texture of the seasoned plywood forms.
5. Prefabricated soffit forming panels (gang forms) are used with conventional falsework systems. These prefabricated soffit forming panels typically consist of an eight-foot (8 ft.) wide and up to 40 ft. long panel comprised of plywood nailed to 2x8 joist. See Figure 1. The use of prefabricated soffit panels have proven to be a reliable, efficient, falsework soffit forming system.
   When the soffit gang form panels are erected onto the falsework stringers they are typically placed with a gap between each panel. This gap aids in the erection and removal of the panel system. This gap is bridged with a form filler panel (filler strip of plywood). See Figure 2.

Figure 1. Typical Gang Form

Figure 3 depicts a typical completed soffit that was formed with gang forms.

Figure 2. Typical Gang Form

Figure 3. Form Filler Strip of Plywood

Building paper shall not to be used to patch cracks or holes in “lost deck” forms. Metal or wood is acceptable provided it does not infringe on the required deck thickness.

Pieces of reinforcing steel may be cast into the interior faces of box girder stems to support “lost deck” forms. When these pieces of reinforcing are used, they should be no larger than a #6, 0.75″∅ bar and shall be at least 1” clear from any permanent reinforcing.

Workmanship

Poor workmanship and materials can lead to undesirable results. Common examples of this are:

1. Loose form filler strips (i.e. not firmly nailed to the joist).
2. Damaged and work panels from overuse.
3. New placement plywood forming sheets next to seasoned sheets.

4. Non-uniform filler strip widths.
5. Non-uniform form line patterns. Skewed bridges exacerbate this.

All of the above can be successfully mitigated with the timely enforcement of this specification.

51-1.03D(4) Construction Joints

Concrete Technology Manual
Additional information regarding girder stem to deck joints can be found in the *Concrete Technology Manual*, Chapter 5 *Concrete Construction*.

Stem to Deck Construction Joint
In 2010, the Structure Maintenance and Investigations (SM&I) group discovered horizontal shear failure between the stem to deck interface in both T-Beam and Box Girder cast-in-place reinforced concrete structures. The shear failure in some cases progressed enough to warrant bridge replacement.

The Division of Engineering Services (DES) Reinforced Concrete Committee evaluated this issue with the objective of increasing horizontal shear capacity at the girder stem to deck interface. In addition to other changes in design practice and procedures, it was concluded that horizontal shear capacity across the stem to deck interface increases significantly when the construction joint is intentionally roughened to minimum amplitude of ¼". To help assure that proper attention is given to the critical construction joint between the girder stem and the deck, the specifications have been amended.

Figures 1 through 3 depict acceptable roughened surfaces. A hand held garden rake was used to obtain the roughened surface in these examples.

Figure 1: Example of Acceptable Roughened Surface.

Figure 2: Example of Acceptable Roughened Surface.

Figure 3: Example of Acceptable Roughened Surface.

During the roughening operation, care should be exercised to avoid the following:

- Excessive dislodging of coarse aggregates when using the roughening tool.
- Floating/trowelling of the top surface of the stem forcing coarse aggregate into the paste and making the surface too smooth.
- Excessive vibration causing the cement paste to rise and cover coarse aggregates.

In addition to the above, it is also extremely important that the surface of the construction joint be abrasively cleaned per the specifications[1], prior to placement of deck concrete. All laitance, curing compound, and loosened particles of concrete must be removed.

A rough clean construction joint can go a long way in assuring the structural integrity throughout the life of the bridge.

[1] 2010 SS, Section 51-1.03D(4), *Construction Joints*

For projects that are using *Standard Specifications* dated earlier than 2010, your attention is directed to the Division of Construction's Construction Procedure Directive (CPD) 10-8, *Stem to Deck Construction Joint*[2] for more details on how to incorporate the modified joint detail into their projects.

If a change order is required, it will be implemented at no additional cost. It has been determined that the desired roughness can be obtained with insignificant additional effort. Furthermore, removal of the requirement to expose the aggregate by blasting strengthens the *no additional cost* determination.

51-1.03E(2) Placing Mortar

Filling Bolt Holes

Mortar used for filling bolt holes may include additives that reduce shrinkage and cracking of the patch and to provide a better bond between the patch and the existing concrete. Mortar additives (shrinkage reducers, water reducers, bonding agents, etc.) are acceptable provided they meet the following requirements:

1. The additive must not have polyvinyl acetate as the active ingredient.
2. The additive must be acrylic based.

For questions on whether a particular additive is acceptable contact your regional Structural Materials Representative (SMR)[3].

51-1.03F(3) Class 1 Surface Finish

Whip Blasting

Refer to the *Concrete Technology Manual*, Chapter 5 for information on *whip blasting*.

Spray-On Finish

Refer to the *Concrete Technology Manual*, Chapter 5 for information on *spray-on finish.*.

Concrete Barriers

The final surface finish of concrete barriers must conform to the provisions of section 83-2.02D(4), *Finishing*, of the Standard Specifications. You are directed to BCM 162-3 for more specific information on finishing concrete barriers.

[2] http://www.dot.ca.gov/hq/construc/CPDirectives/CPD10-8.pdf
[3] http://www.dot.ca.gov/hq/esc/Translab/OSM/documents/smdocuments/StructuralMaterialsRepresentatives.pdf

51-6 Mass Concrete

51-6.01 General

Concrete Technology Manual
Refer to the *Concrete Technology Manual*, Chapter 7, *Caltrans Advancements/High Performance Concrete*, for additional information.

51-6.01C Submittals

Contractor Requested Construction Joint
Mass concrete is typically identified in the contract by the minimum dimension of an element exceeding 7 feet. Contractors may attempt to have the mass concrete provisions waived by proposing a construction joint at a dimension less than 7 feet. Creating a construction joint below the mass concrete dimension does not ensure that the peak curing temperature or temperature differential is adequately controlled. Such proposals must be evaluated in the context of a complete *Thermal Control Plan* (TCP). The placement of construction joints is not at the Contractors' discretion and their introduction requires Structure Design and Structure Construction approval. The Structure Representative must verify that the Contractor adequately addresses all issues related to the TCP.

51-6.01D(2) Temperature Monitoring

Temperature Monitoring - Not Optional
Caltrans' ongoing evaluation of the mass concrete specifications, including the current minimum dimension, is dependent upon the data collected from the mass concrete temperature monitoring requirements. Temperature monitoring requirements are not to be waived.

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90-4.0	09/30/2014	Concrete Mix Design Database
90-5.0	09/30/2014	Concrete Placement During Hot Weather
90-6.0	09/30/2014	Minimizing Early Age Concrete Bridge Deck Cracks
90-7.0		Blank – Moved to Section 51 on 03-30-15
90-8.0		Blank – Moved to Section 51 on 03-30-15

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Structure Construction
Division of Engineering Services

*Denotes the document is a Bridge Construction Bulletin

Concrete Technology Manual

In terms of engineering costs, approximately 50 cents out of every dollar budgeted by the Bridge Department for construction engineering at the project level is spent for field and office activities directly related to concrete and concrete products. In other words, about one-half of our field personnel are at all times engaged in administration and inspection of concrete construction. In view of this, the importance of a thorough and complete understanding of concrete as a structural building material is obvious and requires no elaboration.

This manual was originally intended as a textbook for use in conjunction with the Bridge Department's training course in basic concrete technology. During the editing process it was broadened in scope and content until now it includes all aspects of concrete production and concrete construction practices, as well as materials testing and control procedures.

With the amount of concrete structures placed on the State Highway System the importance of a thorough and complete understanding of concrete as a structural building material is obvious and a detailed discussion of concrete can be found in the *Concrete Technology* Manual on the web at either:
http://dschq.dot.ca.gov/OSCHQDownloads/misc/CTM2013.pdf
or
http://www.dot.ca.gov/hq/esc/construction/manuals/

Control of Cement Content in Concrete

The Standard Specifications (SS)[1] require that the cement content of concrete mixtures be verified in accordance with procedures described in California Test 518. As stated in the SS[2], "…For testing purposes, supplementary cementitious materials (SCM) are considered to be cement. …" Although California Test 518 is titled *Method of Test for Unit Weight of Fresh Concrete*, it also gives instructions for determining the volume of concrete(s) per batch, and determining the cement content (CC_A) in pounds per cubic yard of concrete produced.

Form DS-OS C68, *Worksheet for California Test 518, Unit Weight of Fresh Concrete*, was originally developed by Materials Engineering and Testing Services (METS) and modified by Structure Construction (SC) (Form SC-3702) to facilitate the calculations process to determine that the cement content complies with specification requirements. (Attachments No. 1 and No. 2 are examples of completed Form SC-3702 (formerly form DS-OS C68.)

Unit Weight Testing

For concrete mixes, the goal is to obtain the required cement content in each cubic yard of concrete. If it were practical to accurately weigh all the ingredients required to produce one cubic yard of concrete, mix them thoroughly, and place them in an accurate cubic yard measure, then determining the proper amount of cement would be easy. If the batch which contained the specified amount of cement overflowed the cubic yard measure, it would actually not contain enough cement per cubic yard. If the batch did not fill the cubic yard measure, it would actually contain more cement than required per cubic yard. In the event that the mix contained too much or too little cement per cubic yard, adjustments would have to be made to produce nearly an exact cubic yard containing the specified amount of cement per cubic yard. Since it is not practical to check the cement content by making use of a cubic yard measure, the unit-weight test is used to provide the necessary data needed to make adjustments to the mix design and to make corresponding adjustments to the load weight.

In effect, these two procedures are the same, except that in the unit-weight test, a small sample, that is practical to handle, is used, and the volume produced per load is calculated by simple proportion. The unit-weight test is limited to the determination of the unit-weight of fresh concrete in pounds per cubic foot, but does include equations that may be used for calculating the volume of concrete per load and the actual cement content of the concrete produced.

In reviewing the calculations, only two factors are needed to calculate the volume of the load. The calculation for volume of load is as shown below:

[1] 2010 SS, Section 90-1.01, *General*.
[2] 2010 SS, Section 90-1.01D(2), *Cementitious Material Content*.

W	=	Unit weight in pounds per cubic foot (the net weight of concrete in the calibrated bucket times the calibration factor)
W_t	=	Total scale weight in pounds per load of all the ingredients in the load of concrete.
S	=	Volume of concrete produced in cubic feet per load.

Then $S = W_t/W$

By simple proportion the total weight in pounds-per-load of concrete, divided by its unit weight in pounds-per-cubic-foot, equals the quantity in the load in cubic feet. The important thing to remember is that you must have the actual weights of the water, cement and aggregates going into the load.

It is important to note that the unit-weight test does not check batching accuracy. The Field Engineer must first be assured that scales at the batch plant are accurate by the State Bureau of Weights and Measures inspection and seal. Batching accuracy can then be checked by observing the batching operation and by requiring the Contractor to determine the gross and tare weights on the mixer truck. The gross minus tare weight method should only be used as a rough check of batching accuracy, not as the value to be used in the calculations of batch volume[3].

To determine the cement content of the concrete being produced, the actual weight of cement included at the batch plant will need to be known. The weight of cement should be verified by observation of the scale weights as the batch is being produced. The calculation for cement content is shown below:

*CC	=	Cement content in pounds per cubic yard.
W_a	=	Number of pounds of cement in the load (verified by observation or recording equipment record).
S	=	Volume of concrete produced per load in cubic feet (determined from unit weight as described above).

Then by proportion: $CC/27 = W_a/S$ or $CC + 27W_a/S$

*Note: CC = the actual weight of all cementitious materials.

The cement content in pounds per cubic yard is to 27 cubic feet per cubic yard, as the number of pounds in the load is to the number of cubic feet in the load. The number of pounds of cementitious material in the load, must be the actual amount of cement, as determined by plant scales. Weighing is the only way to know exactly how much cement is in the load.

[3] 2010 Standard Specifications (SS) Section 90-1.02F(3)

When the unit-weight test is to be performed, the actual batching of the load to be checked should be observed and scale weights recorded for use in determination of load volume. The mixer drum should also be checked prior to batching to be sure that a significant quantity of water is not left in the drum and unaccounted for in batch weights. *If you have not verified by observation or been assured by automatic recording batching equipment records that the intended amounts of cementitious material, water and aggregate were actually batched into the mixer, then the subject test and subsequent calculations cannot be used to determine the cement content.*

STATE OF CALIFORNIA . DEPARTMENT OF TRANSPORATION

WORKSHEET FOR CALIFORNIA TEST 518

FORM SC-3702 (Formerly DS-OS C68) (REV. 09/30/14)

JOB STAMP

TEST NO.		TEST BY	Example #1 for BCM 100-2.0	DATE	
MIX NO.		POUR NO.		LOAD CY	7
DELIVERY SLIP NO.			PENETRATION (KELLY BALL)		

F = CALIBRATION FACTOR FOR MEASURE*	2.006	
1. GROSS WT. OF MEASURE, CONCRETE AND COVERPLATE.	98.08	lb
2. WT. OF MEASURE AND COVER PLATE	24.25	lb
3. NET WT. OF CONCRETE SAMPLE (line 1 minus line 2)	73.83	lb
W = UNIT WT. OF CONCRETE SAMPLE (line 3 times F)	148.10	lb/ft^3
W_a = TOTAL WT. OF CEMENTITIOUS MATERIAL PER LOAD, AS BATCHED.	4725	lb
W_{11} = TOTAL WT. OF FINE AGGREGATE #1 PER LOAD, INCL MOISTURE, AS BATCHED.	8673	lb
W_{12} = TOTAL WT. OF FINE AGG. #2 PER LOAD, INCL MOISTURE, AS BATCHED.		lb
W_{C1} = TOTAL WT. OF COARSE AGG. #1 PER LOAD, INCL MOISTURE. AS BATCHED.	12250	lb
W_{C2} = TOTAL WT. OF COARSE AGG. #2 PER LOAD, INCL MOISTURE, AS BATCHED.		lb
W_{W1} = TOTAL WT. OF WATER PER LOAD AS ADDED AT PLANT. (8.33 LBS PER GAL)	2044	lb
W_{W2} = TOTAL WT. OF WATER PER LOAD AS ADDED AT JOB SITE. (8.33 LBS PER GAL)		lb

$$S = \frac{\text{VOLUME OF CONCRETE PER LOAD IN CUBIC FT.}}{W}$$

27992/148.1=

$$= \frac{W_a + W_{11} + W_{12} + W_{C1} + W_{C2} + W_{W1} + W_{W2}}{W} = 189.00 \text{ ft}$$

CY = VOLUME OF CONCRETE PER LOAD IN CUBIC YARD = S/27 =	7.00	yd
CC = CEMENTITIOUS MATERIAL CONTENT IN LB PER CUBIC YARD OF CONCRETE**		
PRODUCED = W_A /cy =	675.00	Lb/Yd

FILE CATEGORY 37

NOTES:

* Refer to *California Test 18*, Step D, *Calibration of Measure,* for calculating the *Calibration F*actor.

**CC = the actual weight of all cementitious material (cement and supplementary cementitious material).

STATE OF CALIFORNIA . DEPARTMENT OF TRANSPORATION

WORKSHEET FOR CALIFORNIA TEST 518

FORM SC-3702 (Formerly DS-OS C68) (REV. 09/30/14)

JOB STAMP

TEST NO.		TEST BY	Example #2 for BCM 100-2.0	DATE	
MIX NO.		POUR NO.		LOAD CY	7
DELIVERY SLIP NO.				PENETRATION (KELLY BALL)	

F = CALIBRATION FACTOR FOR MEASURE*	2.006	
1. GROSS WT. OF MEASURE, CONCRETE AND COVERPLATE.	95.54	lb
2. WT. OF MEASURE AND COVER PLATE	24.25	lb
3. NET WT. OF CONCRETE SAMPLE (line 1 minus line 2)	71.29	lb
W = UNIT WT. OF CONCRETE SAMPLE (line 3 times F)	143.0	lb/ft^3
W_a = TOTAL WT. OF CEMENTITIOUS MATERIAL PER LOAD, AS BATCHED.	5593	lb
W_{11} = TOTAL WT. OF FINE AGGREGATE #1 PER LOAD, INCL MOISTURE, AS BATCHED.	9394	lb
W_{12} = TOTAL WT. OF FINE AGG. #2 PER LOAD, INCL MOISTURE, AS BATCHED.		lb
W_{C1} = TOTAL WT. OF COARSE AGG. #1 PER LOAD, INCL MOISTURE. AS BATCHED.	10024	lb
W_{C2} = TOTAL WT. OF COARSE AGG. #2 PER LOAD, INCL MOISTURE, AS BATCHED.		lb
W_{W1} = TOTAL WT. OF WATER PER LOAD AS ADDED AT PLANT. (8.33 LBS PER GAL)	2015.3	lb
W_{W2} = TOTAL WT. OF WATER PER LOAD AS ADDED AT JOB SITE. (8.33 LBS PER GAL)		lb

$$S = \frac{\text{VOLUME OF CONCRETE PER LOAD IN CUBIC FT.}}{W}$$

27026.3/143=

$$= \frac{W_a + W_{11} + W_{12} + W_{C1} + W_{C2} + W_{W1} + W_{W2}}{W} = 188.98 \text{ ft}$$

CY = VOLUME OF CONCRETE PER LOAD IN CUBIC YARD = S/27 =	7.00	yd
CC = CEMENTITIOUS MATERIAL CONTENT IN LB PER CUBIC YARD OF CONCRETE**		
PRODUCED = W_A /cy =	799.06	Lb/Yd

FILE CATEGORY 37

NOTES:

* Refer to *California Test 18*, Step D, *Calibration of Measure,* for calculating the *Calibration F*actor.

**CC = the actual weight of all cementitious material (cement and supplementary cementitious material).

Removing the Requirements of CA Test 515 from Structural Concrete

California Test 515, *Method of Test for Relative Mortar Strength of Portland Cement Concrete Sand*, (CT 515) is for the purpose of determining the compressive strength developed by mortar using a given concrete sand, in relation to that developed by mortar using Ottawa sand, and indirectly measures the concrete-making properties of the sand being tested.

At the time this test was developed, the Department was responsible for producing a basic mix design and specifying the cement content. To concrete mix designers, the results of CT 515 provided assurance that the sand proposed for use would not result in an excessive amount of water to obtain the required workability. Currently, Portland cement concrete (PCC) is specified by either strength or by cementitious material content and requires the Contractor to propose mix designs and supply cementitious materials to meet those requirements.

Per CT 515, test samples are prepared with a fixed amount of cement and sand, while water is added until a required flow is achieved. This produces different water/cement ratios which affects compressive strength. When manufactured sands are tested, their angularity and surface profiles require additional water to achieve the required flow, lowering the mortar compressive strength results. This can lead to rejection of otherwise suitable sand for PCC.

The concrete industry and specifications have evolved so that PCC produced today is more than a mixture of aggregate, water, and cement. It is more commonly a mixture of aggregate, water, admixtures (i.e. water reducers, plasticizers, etc.), cementitious, and supplementary cementitious materials that are designed for a given strength and workability.

Provided that fine aggregates meet soundness, durability, sand equivalent, and organic impurity tests, and concrete mix designs meet the other specified requirements, the Department is confident that the requirements of CT 515 can be removed from the Specifications. Engineering needs will still be met, and removal of this requirement can produce additional benefits of utilizing the existing aggregate supply.

When compared to natural sands, manufactured sands are generally characterized as having sharp, angular shaped particles. These properties may result in higher water demand, and concretes that are generally hard to pump or finish. Increased angularity of the fine aggregate in a concrete mix may produce concrete that requires extra effort to place, consolidate and finish. These potential issues can be managed with proper proportioning (mix design) of concrete which may include the use of chemical admixtures such as water reducers and superplastizers.

Contracts Using the 2006 Standard Specifications

For contracts using the 2006 Standard Specifications (SS), the specifications[1] requires fine aggregates used in concrete to meet quality requirements and one of those is mortar strengths relative to Ottawa sand (CT 515).

Concrete for structures that was not specified by compressive strength was to contain a minimum amount of cementitious material based upon the element of the structure. The specifications[2] states that if the specified 28-day compressive strength is greater than 3,600 pounds per square inch, the concrete shall be designated by compressive strength and 42 days will be allowed to obtain the specified strength.

The *Construction Manual*[3] requires that the frequency of sampling for compressive strength of concrete for bridges and major structures be 1 set of cylinders for every 300 yd^3 or as required for acceptance. Minimum 1 set per job and class (mix design) of concrete for each day's production of critical structural elements.

Contracts Using the 2010 Standard Specifications

For contracts using the 2010 Standard Specifications, the specifications[4] no longer requires that fine aggregates used in concrete meet the mortar strengths relative to Ottawa sand (CT 515).

The minimum required compressive strength for all concrete elements in structures is specified[5] as follows:

> Except for minor structures, the minimum required compressive strength for concrete in structures or portions of structures shall be the strength specified, or 3,600 pounds per square inch at 28 days, whichever is greater.
>
> If the specified 28-day compressive strength is 3,600 pounds per square inch or greater, the concrete is designated by compressive strength. For concrete with a 28-day compressive strength greater than 3,600 pounds per square inch, 42 days are allowed to attain the strength described.

The specifications[6] requires that if the concrete has a described 28-day compressive strength greater than 3,600 pounds per square inch, or if prequalification is specified, prequalification of materials, mix proportions, mixing equipment, and procedures proposed for use will be required prior to placement of the concrete.

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[1] 2006 SS, Section 90-2.02B, *Fine Aggregates.*
[2] 2006 SS, Section 90-1.01, *Description.*
[3] Http://www.dot.ca.gov/hq/construc/constmanual/
[4] 2010 SS, Section 90-1.02C(3), *Fine Aggregate.*
[5] 2010 SS, Section 90-1.01D(5), *Compressive Strength.*
[6] 2010 SS, Section 90-1.01D(5)(b), *Prequalification.*

Projects advertised after December 1, 2010 includes the 2010 Standard Specifications. Division of Construction, *Construction Manual Procedure Directive (CPD)* 10-14, *Relative Mortar Strength of Fine Aggregate*, provides procedures to allow this change by Contract Change Order if requested by the Contractor.

All Structure Construction personnel should continue to ensure that quality assurance testing for structural concrete is kept current with the requirements of the Construction Manual, i.e. compressive strength tests for every 300 yd^3 or as required for acceptance and that job sampled compressive strength tests are taken on the earliest use of a concrete mix.

Concrete Mix Design Database

The Materials Engineering and Testing Services (METS), Office of Structure Materials has developed a *Concrete Mix Design Check Program* that stores mix designs in a database. The *Concrete Mix Design Check Program* verifies that the materials in the concrete mix design meet contract specifications[1]. As the database is populated, the Structure Construction (SC) Structure Representatives may save time when a concrete mix design submittal duplicates a previously checked concrete mix design. This database will enhance the Department's ability to analyze the effect of using a specific material, e.g., aggregate from a specific source or a manufacturer's blended cement as well as identify common components of mix designs when material research is occurring.

A volume based concrete mix design check program is available for use on the Structure Construction (SC) Intranet[2]. At this time the *Concrete Mix Design Check Program* verifies the concrete mix design meets the contract specifications, but does not perform volume based calculations. The program is being modified to perform volume based calculations, until then, it is necessary to perform use both programs.

When checking concrete mix designs, SC Structure Representatives are to use the *Concrete Mix Design Check Program*. Prior to starting the program, it is helpful to fill out the *Concrete Mix Design Submittal Checklist*, Form SC-4303, as shown in Attachment No. 1. Then proceed as follows:

1. Open the following link: http://onramp.dot.ca.gov/hq/esc/mets/structure_materials/
2. Start the *Concrete Mix Design Check Program by i*nputting the Contract Number (District EA, xxxxxxx4 or Project ID, 10 digits EFIS No.) for your project in the *Projects field* and click *Go*. *If* the EA is not recognized, contact the local Structure Materials Representative for assistance.
3. Click on the *Concrete Mix Designs* tab.
4. Check the list of approved mix designs for that specific project to see if your mix design has already been approved or not.
5. To check a new Mix Design:
6. Enter *j2guest* in the User ID field and enter "guest" in the password field.
7. Click on the *Concrete Mix Designs* tab.
8. Click on the *Add a new concrete mix design for this project bar*.
9. Begin by entering data into fields on each tab and save data before moving to the next tab. Tab subject headings are listed below:

- SSPs	- Cement	- Aggregates	- Admixtures
- Cast in Place	- SCM	- Gradation	- Water

[1] 2010 Standard Specifications, Section 90, *Concrete.*
[2] http://onramp.dot.ca.gov/hq/oscnet/downloads/forms.htm

10. Remember to click *Save* before opening a new information tab.
11. The related equation check results are available by clicking on the last tab, *Results*. A final report is available by clicking on the *View Report* link at the top of the results page. Both documents can then be printed for the project records.

If issues should arise during the use of the concrete mix design program, contact your Structural Materials Representative (SMR).[3]

[3] http://www.dot.ca.gov/hq/esc/Translab/OSM/documents/smdocuments/StructuralMaterialsRepresentatives.pdf

STATE OF CALIFORNIA . DEPARTMENT OF TRANSPTORATION

CONCRETE MIX DESIGN SUBMITTAL CHECKLIST

FORM SC-4303 (REV. 09/30/14)

The mix design check program requires detailed material information that could be overlooked when the submittal is prepared. The review process can be shortened by giving the following checklist to the Contractor at the preconstruction meeting and using the checklist as an initial check when a mix design submittal is received.

A) CONCRETE USAGE	
◯ Yes ◯ No	Is the concrete designated by compressive strength?
◯ Yes ◯ No	If the concrete is for approach slab or bridge deck, is shrinkage data available?
◯ Yes ◯ No	Is certified test data or trial batch test result data available?
◯ Yes ◯ No	Does the data include:
	☐ Date of mixing,
	☐ Mixing equipment and procedures used,
	☐ The size, weight, type and source of all ingredients used,
	☐ Penetration or slump, if applicable,
	☐ The air content,
	☐ The age at testing and strength of all tested cylinders.
B) CEMENT	
◯ Yes ◯ No	On Authorized Material List?
◯ Yes ◯ No	Conforms to ASTM C-150?
◯ Yes ◯ No	If Type II, III, or V Portland Cement does it contain more than .60% by mass of alkali materials?
◯ Yes ◯ No	Autoclave Expansion > .50%?
◯ Yes ◯ No	If Type II Portland Cement does Tricalcium Silicate Content exceed 65%?
C) BLENDED CEMENT	
◯ Yes ◯ No	Blended Cement used? (If No, skip to D)
◯ Yes ◯ No	Blended Cement Materials on Authorized Material List?
D) SUPPLEMENTARY CEMENTITIOUS MATERIALS (SCM's)	
◯ Yes ◯ No	All SCM's on Authorized Material List?
D-1) FLYASH	
◯ Yes ◯ No	Is Flyash Used? (If No, skip to D-2)
◯ Yes ◯ No	Meets AASHTO M295, Class F?
◯ Yes ◯ No	Alkali total (expressed as equivalent Na_2O+0.658 Ka_2) included?
◯ Yes ◯ No	CaO content included?
D-2) ULTRA FINE FLYASH	
◯ Yes ◯ No	Is Ultra Fine Flyash used? (If No, skip to D-3)
◯ Yes ◯ No	Meets AASHTO M295, Class F?
◯ Yes ◯ No	Sulfur Trioxide (SO_3) content included?
◯ Yes ◯ No	Loss on ignition % included?

D-2) ULTRA FINE FLYASH	
○ Yes ○ No	Alkali total (expressed as equivalent Na_2O+0.658 Ka_2) included?
○ Yes ○ No	Particle size distribution included?
○ Yes ○ No	Strength Activity Index included?
○ Yes ○ No	Expansion at 16 days via ASTM C1567 included?
D-3) RAW OR CALCINED NATURAL POZZOLAN	
○ Yes ○ No	Is Raw or Calcined Natural Pozzolan used? (If No, skip to D-4)
○ Yes ○ No	Meets AASHTO M295, Class N?
○ Yes ○ No	Alkali total (expressed as equivalent Na_2O+0.658 Ka_2) included?
○ Yes ○ No	CaO content included?
D-4) METAKAOLIN	
○ Yes ○ No	Is Metakaolin used? (If No, skip to D-5)
○ Yes ○ No	Meets AASHTO M295, Class N?
○ Yes ○ No	Silicon dioxide (SiO_2) and Aluminum Oxide (Al_2O_3) content included?
○ Yes ○ No	Calcium Oxide (CaO) content included?
○ Yes ○ No	Sulfur trioxide (SO_3) content included?
○ Yes ○ No	Loss on ignition included?
○ Yes ○ No	Alkali total (expressed as equivalent Na_2O+0.658 Ka_2) included?
○ Yes ○ No	Particle size distribution included?
○ Yes ○ No	Strength Activity Index included?
D-5) GROUND GRANULATED BLAST FURNACE SLAG (GGBFS)	
○ Yes ○ No	Is Ground Granulated Blast Furnace Slag Used? (If No, skip to D-6)
○ Yes ○ No	AASHTO M302 Grade 100 or 120?
D-6) SILICA FUME	
○ Yes ○ No	Is Silica Fume used?
○ Yes ○ No	Meets AASHTO M307?
○ Yes ○ No	Reduction in mortar expansion included?
E) AGGREGATE	
○ Yes ○ No	Are the coarse and fine aggregates on the innocuous aggregates list?
○ Yes ○ No	Are proposed gradation(s) included?
○ Yes ○ No	Aggregates on Innocuous Aggregates List? (If Yes, reduced "X" allowed in the 2010 *Standard Specifications*, Section 90-1.02B(3), *Supplementary Cementitious Materials*)
E-1) COARSE AGGREGATE	
○ Yes ○ No	Loss via CT 214 included?
○ Yes ○ No	Loss in Los Angeles Rattler included (CT 211)?
○ Yes ○ No	Cleanness Value included (CT 227)?

E-2) FINE AGGREGATE	
○ Yes ○ No	Loss via CT 214 included (*waived if durability index of fine aggregate is 60 or greater)?
○ Yes ○ No	Durability Index via CT 229 included (*only necessary if CT 214 does not meet qualifications)?
○ Yes ○ No	Organic Impurities Results Included (CT 213)?
○ Yes ○ No	Sand Equivalent Included (CT 217)?
F) CHEMICAL ADMIXTURES	
○ Yes ○ No	Chemical Admixture(s) Used (If No, skip to G)
○ Yes ○ No	On Authorized Material List?
○ Yes ○ No	Dosage verified per plans or manufacturer's recommendations?
G) AIR ENTRAINING ADMIXTURE	
○ Yes ○ No	Air-Entraining Admixture(s) Specified or Used?
○ Yes ○ No	On Authorized Material List?
○ Yes ○ No	Dosage verified per plans or manufacturer's recommendations?

Concrete Placement During Hot Weather

(Criteria for weather-related non-working days)

If concrete placement is the controlling item of work, if all of the following applicable practices are employed, and the Contractor cannot produce concrete which complies with the specified temperature; the placement of concrete must not be commenced and a non-working day allowed[1 & 2]:

1. Chipped or crushed ice is used in place of all mixing water, provided that a commercial supply of ice is available within 80 kilometers (50 miles), measured along the highway, of the project. If a commercial supply of ice is not available within 80 kilometers of the project, the coolest available water is utilized for mixing, i.e., well water instead of above ground stored water.
2. Coarse aggregate stockpiles are kept continuously wet to maximize evaporative cooling.
3. Concrete pours are scheduled so that placement of the concrete is completed prior to 12:00 noon.

[1] 2010 Standard Specifications, Section 8-1.05, *Time.*
[2] 2010 Standard Specifications, Section 1-1.07B.2.2.1, *Glossary – working day.*

Minimizing Early Age Concrete Bridge Deck Cracks

One of the primary factors affecting concrete durability is concrete shrinkage and the resultant cracking that serves as a pathway for corrosive materials like de-icing salts. The Standard Specifications (SS) place limits on shrinkage by requiring that the contractor-proposed concrete mix designs for bridge decks and approach slabs include AASHTO[1] T160, 28-day shrinkage test results as part of the mix design. Overall shrinkage in the forms of autogenous and drying shrinkage is collectively limited by specifications to 0.045 percent for bridge decks and 0.05 percent for approach slabs[2]. The review process does not end with the mix design check. The Structure Representative must verify that concrete delivered to the project is consistent with the approved mix, as variations in aggregate gradation, cleanliness value, sand equivalence, cementious material content, and water-cement ratio can significantly increase the amount of shrinkage that will occur after the concrete has hardened.

Cracking that occurs before concrete has set is referred to as plastic cracking. Concrete finishing and curing operations can directly affect the development of plastic cracks; this type of cracking occurs on concrete surfaces as the top layer dries and shrinks quicker than the moist inner concrete. Initially, bleed water rises within fresh concrete until it reaches the surface and evaporates, but as the internal water supply is depleted, the bleed water flow diminishes. Surface drying starts when the evaporation rate exceeds the bleed rate. Usually associated with warm weather concreting, plastic cracking can occur whenever the shrinkage strain exceeds the surface strength. Unless additional moisture is provided to the surface, plastic cracks can appear. Plastic cracks are shallow, usually less than 2 inches deep, irregular in pattern, and spaced about 1 to 3 feet apart. In any 500 sq ft portion of a new deck surface, if there are more than 50 feet of cracks having a width at any point of over 0.02 inch, a Contractor repair action is required per the Standard Specifications[3]. The standard repair for excessive cracking is methacrylate treatment. An example of plastic cracks on the bridge deck surface is shown in the Figure No. 1.

[1] AASHTO is the American Association of State Highway and Transportation Officials.
[2] SS 2010, Section 90-1.02A *General*.
[3] SS 2010, Section 51-1.01D(4)(d) *Crack Intensity*.

Figure No. 1: Example of Plastic Cracks on the Bridge Deck Surface

Chapter 5, Figure 5-22, of the *SC Concrete Technology Manual*, is an Evaporation Rate Nomograph[4], which relates the following environmental factors to determine the evaporation rate:

1. Air temperature.[5]
2. Relative humidity.
3. Concrete temperature.
4. Wind velocity.

When the nomograph originated approximately 50 years ago, bleed water typically replaced surface water until evaporation rates increased to about 0.2 pounds per square foot per hour where surface drying began to occur. The seven precautions listed below were developed to counteract plastic cracking and should be contemplated by the Contractor as part of the concrete placement plan.

1. Ensure aggregate stockpiles are maintained in the saturated surface dry condition.
2. Ensure surfaces coming into contact with fresh concrete, like forms and subgrade, are thoroughly moistened prior to placement.
3. Erect temporary windbreaks to reduce wind velocity over the concrete surface.
4. Erect temporary sunshades to reduce concrete surface temperatures.
5. Cool the mixing water and aggregates in extreme conditions to keep the fresh concrete temperature low.
6. Reduce time between placing and start of curing, by eliminating delays during construction.
7. Use water fogging sprays to maintain high humidity and surface moisture.

Because of advancements in concrete technology, the threshold for precautionary measures can occur at a much lower evaporation rate. For example, if a high range water reducing admixture is

[4] American Concrete Institute (ACI) 308, *Standard Practice for Curing Concrete*
[5] http://dschq.dot.ca.gov/OSCHQDownloads/misc/CTM2013.pdf

used, the water/cement ratio will be reduced and the final set time could be extended; the result is that the amount of water that can bleed to the surface will be reduced, the time when fresh concrete is exposed to plastic cracking is extended, and the risk of plastic cracking is increased. Another example is the situation where an ultrafine supplementary cementitious material (SCM) like silica fume is included in the concrete mix. Ultrafine SCMs block the capillaries that bleed water would normally follow to reach the surface, so the amount of bleed water that reaches the surface is reduced and the risk of plastic cracking is increased.

The Standard Specifications[6] require two curing methods for bridge decks, the water cure and the curing compound method. The water cure specifications require the application of water as a fine mist to maintain bridge deck surface moisture until curing medium is applied. If the curing medium is <u>not</u> being applied, water misting must be used until concrete has set, then the deck must be continuously sprinkled until the end of cure.

For the best results, mist should be finely atomized water which gradually falls to the bridge deck. A good indication that the water spray is appropriately atomized is that the particles descend at approximately 1 foot per second. Misting should be sprayed from an upwind position over the bridge deck, not onto the deck, raising the humidity in the air above the deck. As the spray falls to the bridge deck, the objective is to maintain a surface sheen but avoid runoff, which would erode the surface.

Curing compound is applied to bridge deck concrete after finishing the surface, immediately before the moisture sheen disappears from the concrete surface but before drying shrinkage or craze cracks start to appear.[7] Curing compound is applied at a nominal rate of 150 square feet per gallon, such that there is uniform coverage without any thin areas or surface runs, by a power-operated spraying device. The curing compound is applied to facilitate concrete strength gain during the cure process; it does not preclude the possibility of plastic cracking, as shown in Figure No. 2.

If there is a delay in the placement of curing compound that may lead to surface dryness and cracking, surface fogging is required to keep the surface moist[8]. After applying curing compound, per the water method, the surface must be misted until covered with curing medium[9].

[6] 2010 SS, Section 51-1.03H or 2006 SS, Section 90-7 *Curing Concrete Structures*
[7] 2010 SS, Section 90-1.03B(3) or 2006 SS, Section 90-7.01B *Curing Compound Method*
[8] 2010 SS, Section 90-1.03B(2) or 2006 SS, Section 90-7.01A *Water Method*
[9] 2010 SS, Section 90-1.03B(2) or 2006 SS, Section 90-7.01A *Water Method*

Figure No. 2: Example of Plastic Cracking Occurring After Curing Compound Application.

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MEMO NO.	ISSUE DATE	TITLE
110-1.0	04/27/1987	Electronic Computer Programs
110-2.0	04/02/1997	Required Use of Computer Security Cables
110-3.0	04/02/1997	Computer Hardware Upgrade Policy
110-4.0	04/02/1997	Unauthorized Modifications to OSC Field Computers

RALPH P. SOMMARIVA, Chief
Office of Structure Construction

Denotes the document is a Bridge Construction Bulletin

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Electronic Computer Programs

General

Electronic computer programs have been developed to facilitate the design and layout of bridge structures. Some of these programs are applicable to field construction problems.

Attached is a list of computer programs (Attachment #1) which are in general use by the Office of Structure Construction. The list includes the Bridge Computer Manual index number, the date that the computer program was issued, and the title of the computer program. All of the listed computer programs are included in the Bridge Computer Manual which contains a complete description of the program and instructions for its use.

On the attached list, those computer programs which have most frequent application to field work are identified by an asterisk (*).

Field construction personnel who want to use any of the listed computer programs may obtain a description of the program and instructions for its use by submitting a written request to the Office of Structure Construction, Attention: Lloyd Johnson.

EDP By Wire Transmission

All routine highway and bridge engineering computer services available by mail to and from Sacramento are also available by wire transmission from and to each District Office. Structure Construction personnel whose locations permit ready access to a District Office are invited to take advantage of this service.

Check with the District Data Processing Coordinator to make sure that he/she is familiar with the Job Control Language (JCL) and that a method exists to enter the program in the system. If not, mail the filled in computer input forms to the Office of Structure Construction in Sacramento, Attention: Lloyd Johnson

Computer Service

Any questions concerning computer service should be referred to the Office of Structures Design, Bridge Computer Services Section in Sacramento, phone ATSS 454-9235 or (916) 323-9235.

Field personnel should check to see if a computer program on hand is the current edition before using it.

Filled in computer input forms for bridge programs to be processed in Sacramento should be sent to:

Office of Structure Construction
Attn: Lloyd Johnson
P. O. Box 942874
Sacramento, CA 94274-0001

Lloyd will assign District, Group, Batch, and Problem Number, and will arrange for the programs' run and its return to field personnel.

ELECTRONIC COMPUTER PROGRAMS IN GENERAL USE

BRIDGE COMPUTER MANUAL

Index Number	Date	Program
*2-1	6-79	Traverse and Horizontal Alignment
*2-2	1-72	Vertical Alignment
2-2.1	1-72	Vertical Alignment Plot
*2-3	4-85	Layout Plot
*2-4	3-72	Bridge Deck Geometrics (See Note 1)
*3-1	2-75	Frame System (See Note 2)
3-1.5	11-73	Frame System Update
3-3	1-72	Bent Analysis
3-3.1	1-72	Sent Analysis Supplement #1
3-4	3-72	Tunnel Arch Analysis
4-1	2-73	Section Properties by Coordinates
4-2	3-72	Girder Deflections
5-1	6-81	Column Design by Ultimate Strength
5-2	8-72	General Column Analysis
6-1	11-80	Prestressed Girder Analysis
6-2	8-72	Continuous Prestressed Girder Analysis
7-1	12-72	Composite Girder Design
7-2	3-72	Composite Girder Analysis
7-3	2-73	Composite Girder X-Section, Analysis
*9-1	1-72	Six-Factor Computations
*9-2	5-80	Superstructure Concrete Quantities
*9-3	1-72	Reinforcing Steel

Note 1 -This program may be used to obtain 4-scale contour plots. Refer to Bridge Construction Memo 2-4.0 for additional information.

Note 2 -This program may be used to determine falsework girder deflections and reactions.

*Computer program which has most frequent application to field work.

ELECTRONIC COMPUTER PROGRAMS IN GENERAL USE

BRIDGE COMPUTER MANUAL

Index Number	Date	Program
* 2-1	6-79	Traverse and Horizontal Alignment
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3-4	3-72	Tunnel Arch Analysis
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4-2	3-72	Girder Deflections
5-1	6-81	Column Design by Ultimate Strength
5-2	8-72	General Column Analysis
6-1	1l-80	Prestressed Girder Analysis
6-2	8-72	Continuous Prestressed Girder Analysis
7-1	12-72	Composite Girder Design
7-2	3-72	Composite Girder Analysis
7-3	2-73	Composite Girder X-Section, Analysis
* 9-1	1-72	Six-factor Computations
9-2	5-80	Superstructure Concrete quantities
* 9-3	1-72	Reinforcing Steel

Note 1 - This program may be used to obtain 4-scale contour plots, Refer to Bridge Construction Memo 2-4.0 for additional information.

Note 2 - This program may be used to determine falsework girder deflections and reactions.

*Computer program which has most frequent application to field work.

Required Use of Computer Security Cables

It is Office of Structure Construction policy that the CPU, monitor, and printer be anchored with a plate and cable system to the desk or table. Any less of a procedure places the computer system at risk.

The recipient of the computer system will be held accountable for protecting the computer equipment assigned to them and this requires (as a minimum) the anchoring of the computer system components to the desk or table top as described above. It is prudent that this anchoring be completed the same day the computer is received or is moved into a new office.

Please verify that OSC computers in your office have been properly secured in conformance with this memo. If security parts are missing, (moving, replaced components etc.) please contact one of the persons listed below for replacement parts.

The security cable system consists of four adhesive plates, a 6 foot cable, special washer and a padlock. An adhesive anchor plate must be installed on each computer component (CPU, monitor, printer) and the back or bottom of the desk. The computer system should be so arranged that a single cable connects all four plates together.

Replacement parts may be obtained by contacting either the OSC system specialist at (916) 227-8401 or the OSC computer operations engineer at (916) 227-8980. Please provide your name, the Caltrans No. (gold CT No. tag) of the CPU and what parts are missing. Order only for your current needs.

Computer Hardware Upgrade Policy

Computer hardware upgrades are not permitted, with minor exceptions, to OSC issued computers. Authorized upgrades must have written approval in advance of the work being performed from the OSC Computer Operations Engineer, (916) 227-8980.

Unauthorized Modifications to OSC Field Computers

Routine system checks have uncovered that a number of unauthorized modifications are being made to individual PCs assigned to OSC field offices. These have included hardware modifications, reconfigured system commands or files, over-written set up files that make the system inoperative and assorted *tinkering* that make the supplied system nearly unrecognizable. In some cases, new personnel have assumed excessive freedom and customized the PCs.

Also discovered have been illegal installation of non-licensed software on the OSC system, and illegal transfer of OSC system software to other PCs in the field office.

These unauthorized/illegal modifications have compromised our system's ability to do the work it was designed to do. The benefits of standardization to multi-users are lost, and computer support is made difficult. Other PCs have required complete reformatting of the hard drive and re-installing the entire software system.

OSC policy for PC use is as follows:

- Use the system as supplied.
- Properly install the updates issued to correct errors and modify programs.
- Do not modify the system configuration.
- Do not install illegal software, or software from other computer systems.
- Do not install copyright software from the OSC system to outside systems.
- Request help and assistance for special situations directly from the System Specialist at (916) 227-8401.

Department policy on illegal use of software is per attached memo dated May 12, 1989 and signed by Deputy Director Carolyn Peirce Ewing.

Individuals who cannot or will not follow policy and basic common sense will have their OSC PC system reassigned to someone else and face potential disciplinary action.

Attachment

State of California
DEPARTMENT OF TRANSPORTATION

California State Transportation Agency

Memorandum

Flex your power!
Be energy efficient!

To: ALL EMPLOYEES

Date: May 12, 1989

File: (leave blank if not used)

From: **DEPARTMENT OF TRANSPORTATION**
DIRECTOR'S OFFICE

Subject: **ILLEGAL USE OF MICROCOMPUTER SOFTWARE**

Software sold for use on microcomputers is copyright-protected in most cases. When it is purchased by a Caltrans employee using Caltrans funds, we honor that protection by not making copies of the product. Software providers are aggressively filing lawsuits against those who duplicate protected software.

In a recent court case filed against the California State Colleges and Universities System, a software company claimed that their copyright protected software was being copied on State microcomputers without separate purchase and license. The State contended that a State agency cannot be prosecuted using Federal Law as the basis of the suit, and the State won the case. On appeal, higher courts found again for the State's position. What this means is that the software vendor is now pursuing the individual professors <u>and</u> their managers for personal liability.

The California State Administrative Manual cites the following:

> "In the conduct of their operations and in the accomplishment of the policies stated above, State agencies and their employees shall employ information technology in a legal and ethical manner consistent with government statutes, rules and regulations. Information technology shall not be used for purposes that are unrelated to the agency's mission or that violate State or Federal law. Contract provisions, including software licensing agreements, shall be strictly followed."
> (S.A.M. Section 4820 - ETHICS)
>
> "Management responsibility for the use of each personal computer, as well as for the security of data, hardware and software, resides with the individual managers who are otherwise responsible for the personnel who regularly use the computer."
> (S.A.M. Section 4990.1 - Management Responsibility)
>
> "Software license agreements must be strictly adhered to. Proprietary software cannot be duplicated, modified or used on more than one machine, except as expressly provided for in the manufacturer's license agreement."
> (S,A.M. Section 4990.1 - Software License Agreements)

Caltrans has added the phrase "...or other modified agreements negotiated with the software manufacturer. "... to this last policy.

The above information was provided to Caltrans Executive Management in a February 17, 1989 letter from Marlin Beckwith, Investigations pursued by some of those managers indicated that in some instances software may have been copied.

All managers, supervisors and employees should investigate their own work environment for potential copyright abuses and assure complete compliance with State regulations. After July 15, 1989, violations of microcomputer software copyrights or State regulations will result in disciplinary action being taken by the Department.

Carolyn Peirce Ewing
Deputy Director
Administration and Transportation
Programs

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MEMO NO.	ISSUE DATE	TITLE
112-1.0	09/02/1974	Longitudinal Deck Construction Joints
112-2.0	11/01/1982	Testing Bridge Deck Surfaces for Compliance with the Straightedge or Profilograph Requirements
112-2.1*	09/15/1999	Profile Counts of Deck Profilographs
112-3.0	09/02/1974	Supports for Finishing Machines
112-4.0	06/17/2014	Friction Testing of Bridge Decks
112-5.0	06/17/2014	Methacrylate Deck Crack Treatment
112-5.1*		(Blank – Removed 06-17-14)
112-5.2*		(Blank – Removed 06-17-14)
112-6.0	05/27/2011	Quieter Bridge Deck Construction

ROBERT A. STOTT
Deputy Division Chief
Structure Construction
Division of Engineering Services

**Denotes the document is a Bridge Construction Bulletin*

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Longitudinal Deck Construction Joints

Expanded Metal Lath Forms

Expanded metal lath may be used to form longitudinal deck construction joints provided the joint is constructed in compliance with the following requirements.

1. A continuous wood strip, the height of the bottom steel clearance, shall be securely fastened to the deck form.

2. Another continuous wood strip, the height of the top steel clearance, shall be fastened securely on top of the top steel. This strip shall be set to grade and held firmly in place by means independent of the reinforcing steel.

3. The expanded metal lath shall be securely fastened between these wood strips.

4. Concrete along the face of the metal lath form must be cured by the water method.

5. Before the second pour is made, wood strips must be removed and any fractured concrete removed. Metal lath that is firmly embedded in concrete may remain in place provided it has the same clearance from the concrete surface as the specified steel clearance. Any loose mesh must be removed.

Testing Bridge Deck Surfaces for Compliance with the Straight Edge or Profilograph Requirements

General Information

All deck surfaces must be tested to be assured of compliance with the Bridge Deck Finishing Specifications. Note that the Specifications have the following requirements:

1. Concrete decks or concrete approach slabs, which are to be covered with one inch or more of another material, will be tested both longitudinally and transversely with a 12-foot long straightedge.

2. The completed roadway surfaces of structures, approach slabs, and the adjacent 50 feet of approach pavement will be tested with a bridge profilograph in the longitudinal direction, and with a 120-foot straightedge in the transverse direction.

Note that if a concrete deck were specified to have a two inch AC overlay, it would be necessary to test the concrete deck both longitudinally and transversely with a 120-foot straightedge prior to placing the AC overlay; and then after placing the overlay, the AC surface should be tested longitudinally with a bridge profilograph and transversely with a 120 -foot straightedge.

Straightedging Deck Surfaces

When the Specifications require the use of a straightedge to check deck surfaces, the decks must be systematically checked with a 120-foot straightedge over the entire area. This should be done as soon as the concrete can be walked upon without damaging the deck surface, and should be completed prior to the time that the deck surface is covered with rugs, mats, or other material that would interfere with the straightedging operations. Any places which do not meet specifications should be marked with red spray paint.

It is the Contractor's responsibility to straightedge the deck while the concrete is wet. Structure Construction personnel should not straightedge wet concrete except under unusual circumstances.

Profilograph Testing of Deck Surfaces

When the Specifications require the use of the Bridge Profilograph to check deck surfaces, the deck will be tested with the profilograph in accordance with the Test Method No. Calif. 547. High points in excess of 0.25 inch should be marked with red spray paint. It shall be the Contractor's responsibility to schedule the profilograph testing operations. The Contractor shall

request testing at least 7 days prior to need, and shall ensure that the entire area to be tested has been cleared and cleaned of all obstructions.

Because the Districts currently have the expertise in maintaining highway profilographs, they will also have the responsibility for storing and maintaining the bridge profilographs. Therefore, when bridge profilographs are needed, they should be obtained from the District Construction or Materials Department. The method of handling the profilographs varies somewhat in the various Districts. It will therefore be necessary for the Structure Representative to become familiar with the procedures, and persons to contact, in the District in which he is working. When not actually in use on a project, the profilograph should be returned to the proper District authority.

Letter to Contractor

As soon as possible after testing the deck with the profilograph and/or the straightedge, a letter should be written to the Contractor advising him that the deck has been checked for compliance with the profilograph requirements and/or the straightedge requirements. The letter should describe the specific locations that fail to meet the straightedge specifications, or describe any deficiencies in meeting the profilograph specification. The letter should state that the specific deficiencies must be corrected before the contract can be accepted. After the deficiencies have been corrected, or if the entire deck initially complies with the applicable straightedge or profilograph requirements, then write the Contractor a letter stating that the deck was checked and that it complies with the profilograph requirements and/or the straightedge requirements whichever is applicable. (See Attachment No. 1 for a sample letter relative to concrete decks or concrete approach slabs which are to be covered with one inch or more of another material. See Attachment No. 2 for a sample letter relative to the completed surfaces of bridge decks, approach slabs, and adjoining 50 feet of approach pavement.)

The Specifications allow the Engineer to point out a contract deficiency to the Contractor at any time. However, once the Engineer has completely satisfied himself that the deck surface complies with the Specifications, and has given the Contractor a letter advising him of this; it is the mark of an ethical Engineer to consider the matter closed.

(Sample of letter to be sent to the Contractor for concrete decks or concrete approach slabs which are to be covered with one inch or more of another material.)

The finished surface of the deck concrete at_____ Bridge No._____ has been tested for compliance with the straightedge requirements in Section 51-1.17, *Finishing Bridge Decks*, of the Standard Specifications.

(USE EITHER)

All areas tested were found to comply with the specified straightedge requirements.

(OR)

Areas that do not meet the straightedge requirements have been marked, and are located as noted below:

(EXAMPLES) Sta. 300+52 (5 ft. to 15 ft. from RT. EOD)

Sta. 301+60 (10 ft. from Rt. EOD)

Hinge in Span 3 (Entire bridge width)

Transversely across longitudinal construction joint. (Sta. 300+10 to 302+10)

These deficiencies must be corrected before the _____ overlay is placed.
(describe overlay)

Notification shall be given to the Resident Engineer prior to performing the corrective action.

(Sample of letter to be sent to the Contractor for completed surfaces of bridge decks, approach slabs, and adjoining 50 feet of approach pavement.)

The completed surface of (bridge deck) (approach slab) (adjoining 50 feet of approach pavement) at Bridge No. _____, Bridge Name _____ has been tested for compliance with the profilograph requirements and the transverse straightedge requirements of the Standard Specifications.

(USE EITHER)

All areas tested were found to comply with the specified profilograph (and) (or) the transverse straightedge requirements.

(AND/OR)

The profilograph trace indicates that there are high points in excess of 0.25 inch and that the profile count exceeds 5 per hundred feet. High points in excess of 0.25 inch have been marked with red spray paint. A profile trace is available for your examination at the Resident Engineer's office. The completed surface must be ground in accordance with the requirements in Standard Specifications Section 42, *Groove and Grind Pavement*, until the specified smoothness tolerances are met.

(AND/OR)

Straightedging in a transverse direction indicated that the roadway surface varied more than 0.02 foot from the lower edge of a 120-foot long straightedge at the following locations: Areas that do not meet the straightedge requirement have been marked, and are located as noted below:

(EXAMPLE) 4 foot from the left EOD between Sta 300+00 and Sta. 300+75.

Longit. const. jt. at center of left bridge between Sta. 300+50 and Sta. 301+10.

These deficiencies must be corrected before the contract can be accepted. Notification shall be given to the Resident Engineer prior to performing the corrective action.

B99-09

File	**BCB 112-2.1**
Date Effective	09/15/99
Expiration Date	None
Supersedes	None
Approved by	R. P. Sommarive, Chief Division of Structure Construction

Subject: Profile Counts of Deck Profilographs

This Bulletin clarifies references to 'profile count' in Section 51-1.17, *Finishing Bridge Decks*, of the Standard Specifications. The **1992 edition** of the specification states that:

> The surfaces shall have a profile trace showing no high points in excess of 0.25-inch and the portions of the surfaces within the traveled way shall have a profile count of **5 or less in any 100-foot section**.

This "profile count" refers to the summation of the height of scallops that are greater than 0.03" high and measured to the nearest 0.05" and expressed in tenths of inches. Thus, a count of 5 would reflect 0.5" of scallops recorded in a 100 ft section of profile trace. This is outlined in California Test 547.

The **1995/1999** editions of the specification states:

> The surfaces shall have a profile trace showing no high points in excess of 6.35 mm, and the portions of the surfaces within the traveled way shall have a profile count of **13 or less in any 30-m section**.

This profile count refers to the same criteria as listed above for the 1992 specification with the exception that the value is expressed in millimeters. Thus, if you had a count of five after following the procedure in California Test 547, you would multiply this value by 2.54 to obtain the "metric" value of the profile count.

Use the templates required by California Test 547 and perform the evaluation in accordance with the test method. When reporting the "count", you **multiply the count** (which represents tenths of an inch in a hundred feet) **by 2.54** to obtain the "count" to compare to the metric specification reference (i.e. a count of 7 using the templates would equate to a metric profile count of 17.8).

c: BCR&P Manual Holders
Consultant Firms
BFelker, Construction Program Manager

"Providing the technical expertise for quality built structures"

Supports for Finishing Machines

RAILS

The Standard Specifications require that "Rails for support and operation of finishing machines or hand-operated strike-off devices shall be completely in place and firmly secured for the scheduled length for concrete placement before placing of concrete will be permitted."

"Rails" as used in this specification shall be interpreted to be the continuous structural members which are readily adjustable for elevation, and which support the finishing equipment. These "Rails" must be in place for the entire length of the pour.

Various devices may be placed on top of these rails for purposes of wearing or spacing. Such devices do not have to be full length and may be "leap-frogged" around the finishing equipment.

Friction Testing of Bridge Decks

After deck surfaces and approach slabs have been textured, the Engineer tests the coefficient of friction of the concrete surfaces under California Test 342[1]. Refer to *Standard Specifications*[2] for coefficient of friction testing requirements.

Newly Constructed Bridge Decks

Coefficient of friction testing should be performed on each deck. The test must be performed at a location which is representative of that portion of the deck surface exhibiting the lowest coefficient of friction. Once the representative area has been tested and shown to meet the specification, more tests will not be required unless, in the opinion of the Structure Representative, the test results are not representative of the bridge deck skid resistance. Skid resistance testing of small bridge decks and approach slab replacements may be waived provided the Structure Representative makes a visual inspection of the deck surface and determines that the surface may reasonably be expected to have a coefficient of friction of 0.35 or greater as required by the Standard Specifications. This inspection must be documented and included in the job records.

Rehabilitated Bridge Decks

The *2010 Standard Specifications* requires coefficient of friction testing for projects with bridge deck methacrylate treatment[3] and/or polyester concrete overlays[4]. These specifications were revised in March 2014 to include additional requirements and should be in projects with bid opening dates after April 24, 2014. This revision will require that the test area or trial overlay be authorized prior to the start of production work and then be used as a standard of comparison for the production work. Using this comparative process will allow Caltrans to better use the available friction testing resources.

Bridge Deck Methacrylate Resin Treatment

Changes to the *Standard Specifications*[5], include the following:

- The Contractor is to notify the Engineer at least 15 days before treating the test area.
- Test area demonstrates:
    1. Compliance with the specifications.
    2. That the work will be completed within the time allowed.

[1] Method of Test for Surface Skid Resistance with the California Portable Skid Tester
[2] 2010 Standard Specifications (SS), Section 51-1.01D(4)(c), *Coefficient of Friction*.
[3] 2010 SS, Section 15-5.05, *Bridge Deck Methacrylate Resin Treatment.*
[4] 2010 SS, Section 15-5.06, *Polyester Concrete Overlay.*
[5] 2010 SS, Section 15-5.05, *Bridge Deck Methacrylate Resin Treatment*.

- The Engineer performs friction testing of the treated test area. Ten days will be allowed after completion of the test area for the Engineer to perform the testing.
- Test area must be authorized before the Contractor starts deck treatment activities.
- The authorized test area will be the standard of comparison in determining the acceptability of treated deck surfaces.
- The Engineer may perform testing under California Test 342 to verify the coefficient of friction of the treated deck surfaces.

The authorized test area will provide a standard of comparison for the Structure Representative to make a visual inspection between the treated test area and the production work for acceptance. The Structure Representative may perform additional testing to verify the coefficient of friction if the Contractor's materials, methods, or procedures have changed from those used on the test area.

There are additional specification requirements that need to be met before authorizing traffic or equipment onto the treated deck surface (overlay). See BCM 112-5.0, *Methacrylate Deck Crack Treatment*, for additional information.

See *Scheduling Skid Testing* below.

Polyester Concrete Overlay

Changes to Standard Specifications[6], include the following:

- The Contractor is to notify the Engineer at least 15 days before constructing the trial overlay.
- Trial overlay demonstrates:
    1. Compliance with the specifications.
    2. That the work will be completed within the time allowed.
- The Engineer performs friction testing of the trial overlay. Ten days will be allowed after completion of the trial overlay for the Engineer to perform the testing.
- Trial overlay must be authorized before the Contractor starts production overlay activities.
- The authorized trial overlay will be the standard of comparison in determining the acceptability of polyester concrete overlay.
- The Engineer may perform testing under California Test 342 to verify the coefficient of friction of the polyester concrete overlay.

The authorized trial overlay will provide a standard of comparison for the Structure Representative to make a visual inspection between the trial overlay and the production work for acceptance. The Structure Representative may perform additional testing to verify the coefficient of friction if the Contractor's materials, methods, or procedures have changed from those used on the trial overlay.

Trial overlays should be constructed at a location within the project limits that will facilitate testing and inspection by the Engineer and also simulate climate and weather conditions. The depth/thickness of the concrete base for the trial overlay must have the size, strength, and load

[6] 2010 SS, Section 15-5.06, *Polyester Concrete Overlay*.

capacity to accommodate the equipment used for the work. A minimum 4″ slab thickness is recommended. After acceptance of all polyester concrete overlay surfaces the trial overlay and concrete base must be disposed of.

See *Scheduling Skid Testing* below.

Construction Procedure Directive (CPD)

The Division of Construction issued CPD 14-4 to incorporate the revised specifications for coefficient of friction testing at bridge deck treatments[7] into all projects where applicable. CPD 14-4 provides information needed to implement the revised specification for ongoing projects without the revised specification where methacrylate deck treatment or polyester concrete overlay work has not commenced.

For projects where you are able to implement CPD 14-4, follow the guidelines *Rehabilitated Bridge Decks* above.

If you are unable to incorporate the specification change from CPD 14-4, the Engineer should require a minimum of one friction test per project to ensure that the minimum coefficient of friction of 0.35 has been achieved. The Engineer will use the tested bridge deck as a standard of comparison to determine compliance with the applicable Standard Specifications for the remaining bridge decks.

Scheduling Skid Testing

To meet the 10-day test window requirement, and to ensure that there are no delays to the contract, the tests will have to be scheduled as soon as possible. Between the 15 day notification by the Contractor prior to performing the test area or trial overlay, and the 10 day allowance to perform the testing, this should be sufficient time to schedule and complete the test.

Coefficient of friction testing can be arranged by contacting the appropriate staff listed on the instruction tab of *Request for Portable Skid Test* form available at:
http://onramp.dot.ca.gov/hq/oscnet/downloads/forms.htm [*Skid Test Request* Form (METS)]

[7] 2010 SS, Section 15-5.05, *Bridge Deck Methacrylate Resin Treatment* and 2010 SS, Section 15-5.06, *Polyester Concrete Overlay.*

Methacrylate Deck Crack Treatment

Methacrylate deck crack treatment uses methacrylate resin to seal cracks in concrete bridge decks and is designed to be a crack sealer only. It is not a surface overlay. Methacrylate resin is used to repair newly placed decks that exceed the crack intensity limits and to rehabilitate existing bridge decks that have deteriorated over time.

Typically, the resin is applied to the deck surface by hand or mechanically sprayed and spread with a broom or squeegee. In general, the application of methacrylate on a bridge deck is a simple process. However, careful inspection is needed to assure the treatment is effective and that the treated surface maintains a desirable roadway surface condition.

Attachment No. 1 contains inspection guidelines to assist Structure Construction personnel when inspecting a methacrylate application.

Additional information regarding methacrylate is available in the *Concrete Technology Manual*.

Methacrylate Deck Treatment Inspection Guidelines

Prior to Starting Work

- Review Standard Specification[1].
- Review and authorize the Contractor's program for public safety associated with the use of methacrylate resin as required in the Special Provisions.
- Forward Form CEM 3101, *Notice of Materials to be Used*, to Materials Engineering and Testing Services (METS).
- Verify the methacrylate was tested and released by METS (Form TL-0101, *Sample Identification Card*).
- Verify the sand and absorbent material meets the specification[2] requirements.
- Refer to BCM 112-4.0, *Friction Testing of Bridge Decks*, for instructions on obtaining verification of the coefficient of friction for the test area.
- Hold a meeting with the Contractor to discuss the required test area, skid testing, application equipment, safety, abrasive cleaning methods, and a contingency plan if the resin does not cure in time.

During Construction Operations

- Prior to treating the required deck areas, ensure that the methacrylate deck treatment has been performed on a test area and the results are accepted. The main purpose of conducting a test area is to obtain the necessary information (i.e., application rate, initiator/promoter amount, set time, coefficient of friction, etc.) to assure that the work within the traveled way can be completed without disruption to the traveling public.

- The concrete deck surface must be cleaned prior to methacrylate application. Steel shot blasting is specified to prepare the deck surface. If the deck surface becomes contaminated before the methacrylate is applied the deck surface must be cleaned again[3]. Check the contract Special Provisions for other specified deck cleaning methods, if any.

- Prior to applying the methacrylate, weather conditions and deck surface temperature should be checked. Current specifications limit the relative humidity to 85 percent or lower at the time of treatment. The deck surface needs to be dry and between 50 to 100 degrees F.

- Methacrylate resin can be applied by hand or with mechanical equipment. If mechanical sprayers are used they must be the *airless* type. Compressed air spray application creates

[1] 2010 Standard Specifications (SS), Section 15-5, *Bridge Rehabilitation* and Section 51-1.01D(4)(d), *Crack Intensity*.
[2] 2010 SS, Section 15-5.05B, *Materials*.
[3] 2010 Standard Specifications (SS), 15-5.01C(2), *Prepare Concrete Deck Surfaces*, or 2006 Special Provisions (SP), *Clean Bridge Deck*.

mist and cannot be used. Typical application involves hand placing with buckets and spreading with a squeegee or broom.

- The resin must be placed within 5 minutes of the initiator being mixed. Excess material needs to be distributed (by a squeegee or broom) within 10 minutes of resin placement. The redistribution of the excess material is important to ensure cracks are completely filled and to avoid thick glassy spots that may reduce the coefficient of friction. Spread rates are specified in the contract specifications[4]; the exact spread rate is determined by the Engineer and will vary depending on the surface condition, aggregate type, and roughness of the deck surface. The specifications[5] suggest an application rate of 90 ft^2/gal. As a *rule of thumb*, 100 ft^2/gal (2.45 m^2/liter) is a good starting point for normal concrete. Lightweight concrete will require more resin and may reduce the spread rate to about 65 ft^2/gal (1.6 m^2/liter). For very dense concrete, less resin is required and the spread rate can increase to about 175 ft^2/gal (4.29 m^2/liter). As a *rule of thumb* the surface above cracks should be slightly wet with resin, 20 minutes after application. This is an indicator that the cracks are completely filled. If dry after 20 minutes, increase the amount of resin being applied. If ponding is evident, reduce the amount of resin. It is essential that the resin remains fluid long enough (40 to 90 minutes) for the cracks to be filled. If rapid gelling occurs the material should be rejected.

- Methacrylate resin must only be applied to the deck area. The Contractor is required to protect or avoid placing resin on other parts of the structure (e.g., barrier rails, joints, drainage facilities, etc). It is important to ensure resin does not leak or drip into waterways, roadways, or parking areas below the bridge. Sealing of joints and scupper drains is one method to prevent this.

- Sand is applied to increase skid resistance. Careful inspection of the deck surface after the sand application is needed to assure that the sand adheres to the deck. Any areas found absent of sand adhesion must be abrasively blasted. Vacuum attachments must be used during abrasive blasting operations.

- Apply absorbent material. The absorbent material removes oily residue that can form and prevents tracking of residue onto the adjacent pavement.

Prior to Opening the Treated Area to Traffic, the Following Requirements Must be Met

1. The treated surface is tack free and not oily.
2. The applied sand cover adheres and resists brushing by hand.
3. Excess sand and absorbent material has been removed.
4. No material will be tracked beyond the limits of treatment by traffic.
5. The treated deck should be comparable to the test area.

Typical Problems Associated with Bridge Deck Methacrylate Resin Treatment Operations

[4] 2010 SS, Section 15-5.05C, *Construction*, or 2006 SP, *Bridge Deck Methacrylate Resin Treatment*
[5] 2010 SS, Section 15-5.05C, *Construction*, or 2006 SP, *Bridge Deck Methacrylate Resin Treatment*

- Oiliness–The tack, or the oiliness, of methacrylate resin can create serious problems, especially in cold night closures. Opening traffic lanes prior to the complete cure of the resin can cause the tracking of residue, oiling of cars, and/or reduced skid resistance. This issue is due to o*xygen inhibition* of the top surface. Methacrylate resin cures from the lack of oxygen, thus the exposed surface tends to cure last. Even if the bulk of the resin sets up and can resist penetration with a screwdriver, the surface can still be covered with an oily sheen. Modern methacrylate resins contain additives to prevent this phenomenon.

- Inability to spread material - Heat and sunlight can cause methacrylate to set faster. Occasionally, the resin will set before the material is spread. This causes the worst case for crack sealing as it prevents the resin from properly flowing into the cracks.

- Sand does not adhere–Resin that sets prior to applying sand will result in the creation of glassy spots. The glassy areas may have reduced skid resistance and remedial work to repair these areas would be required. Methods that have been proven effective are to abrasively blast the glassy areas. For larger areas where the sand was not promptly applied and didn't adhere, resin and sand can be reapplied (time permitting).

Quieter Bridge Deck Construction

Background

Quiet pavement strategies produce traffic noise reduction benefits over time without compromising safety, ride quality, and durability of pavement surfaces. The noise emitted from the highway system has become a subject of complaint and environmental impact to residents, specifically to those in urban areas. The primary traffic sources of generated noise are classified into three categories: propulsion, aerodynamics, and tire/pavement interface. At highway speed the dominant noise is generated at the tire/pavement interface. Engineers throughout the world have been researching methods to reduce the noise impacts of highway systems beyond building sound barriers. These methods include improvement of the roadway pavement types and textures to reduce tire/pavement interface noise.

The standard bridge deck texture method used by California contractors is transverse texturing which has proven to be significantly louder than longitudinally tined Portland Concrete Cement (PCC) pavement measured by the On-Board Sound Intensity (OBSI) method. Experience has shown that contractors aggressively texture the bridge deck to ensure meeting the specified minimum coefficient of friction value. Aggressive texturing often results in a uniformly unbalanced or uneven surface known as *shingling* which further increases the vehicle tire noise.

Quiet Pavement Research (QPR) has shown that traffic noise can be minimized by incorporating quiet pavement strategies in construction practices at little or no additional cost. QPR has found that for rigid pavement including bridge decks *longitudinal grinding and grooving*, or *longitudinal tining* are two textures that can be used to reduce tire noise. Research has found that these two methods produce less tire/pavement interface noise than the transverse texturing method currently used on most California bridge decks. The texturing is almost identical to that used on PCC pavement highways. Tire noise measured by the OBSI method on California bridge decks that are transversely textured range from 105 decibels (dB) to 112 dB. For comparison, tire noise measurements for *longitudinal tining* range from 103 dB to 105 dB, for *longitudinal grinding and grooving* range from 100 dB to 103 dB, and for *flexible pavement* range from 95 dB to 105 dB. An increase in 10 dB is perceived as double the noise to the human ear. Grinding has reduced the tire noise on bridge decks by as much as 10 dB and has been used as an interim measure to remedy noisy transversely textured bridge decks.

Longitudinal grinding and grooving of PCC roadways and bridge decks produce adequate coefficient of friction results for bridge decks. Grinding is typically used to remedy surfaces that do not meet friction requirements. The primary purpose of the longitudinal grooving applied to the ground surface is to increase water channeling below the tire.

The PCC pavement Standard Specification Section 40-1.10, *Final Finishing*, requires an initial texture created with a burlap rug or broom device which produces striations parallel to the centerline. The purpose of the initial texture is to slightly roughen the su*rfac*e to achieve the required friction. This is also necessary to achieve the required friction on longitudinally tined bridge decks. Similar to grooving, the primary purpose of *longitudinal tining* is to increase water channeling below the tire.

There are many polyester concrete bridge decks in service on the state highways today that have been tined longitudinally. The longitudinally tined polyester concrete bridge deck surfaces have resulted in lower than expected tire noise compared to a transversely tined bridge deck and have easily exceeded the minimum friction requirement. Figure No. 1 is an example of a longitunally tined polyester concrete bridge deck surface.

For more information on quieter pavement technology see the report on the Offices of Structure Construction Website *Bridge Deck Tire Noise Research* at the following intranet link:

http://dschq.dot.ca.gov/OSCHQDownLoads/misc/QP_Report_120109.pdf

Current Practice

Bridge decks are textured in accordance with Standard Specification Section 51-1.17, *Finishing Bridge Decks*, which addresses smoothness, friction, and crack intensity with no limitation for tire noise. The contractor chooses the means and method to achieve the specification requirements that are defined by:

- An upper limit for smoothness and crack intensity.
- A minimum friction value.

No practice currently exists to limit tire noise on bridge decks.

New Practice

All new bridge deck projects advertised after January 1, 2011 include the Standard Special Provision (SSP), *Bridge Deck Surface Texture*. This specification provides the following two options for bridge deck texturing:

1. Longitudinal grinding and grooving.
2. Longitudinal tining.

The requirements for bridge decks which address smoothness, friction and crack intensity remain unchanged.

A longitudinal tined texture can easily be accomplished on a bridge deck. Longitudinal tining machines are commercially available but are not mandatory. The longitudinal tining machine will accommodate a bridge deck with a variable cross slope, a crown, or superelevation. The contractor may propose another technique or device to achieve the requirements of the specification.

Following are construction aspects that should be ensured when using the *longitudinal grinding and grooving* option and the *longitudinal tining* option:

***Longitudinal Grinding and Grooving* Option:**

- The bridge deck thickness will be increased ¼ inch.
- The concrete mix design must meet the specification requirements for cementitious material and quality aggregates to ensure the deck surface durability.
- The bridge deck drains and other permanent fixtures should be set to the final grade per the contract plans.
- All recessed areas that will not be accessible by the grinding blades such as the area adjacent to bridge deck drains should be hand textured longitudinally to match adjacent concrete, while the concrete is wet.
- The 18 to 20 inches of bridge deck surface adjacent to the barrier rail will be inaccessible by the grinding blades. This area should be hand textured longitudinally to match adjacent concrete, while the concrete is wet.
- Figure No. 2 is an example of a bridge deck surface using the *longitudinal grinding and grooving* option.

***Longitudinal Tining* Option:**

- The contractor must drag burlap or a light broom longitudinally in advance of the tining operation to ensure that the surface friction is adequate. Attention should be paid to the texture operation to ensure the burlap or other tools used to roughen the surface in front of the tining operation are evenly weighted and produces a flat roughened surface.
- The concrete mix design must meet the specification requirements for cementitious material and quality aggregates to ensure the deck surface durability.
- The concrete mix design water content and corresponding slump is important to ensure the specified tining texture can be achieved and is consistent.
- Ensure that the concrete will be delivered timely per the specifications. It is desirable to place as much deck as possible for each longitudinal tining pass to minimize starts and stops in the tining pattern.
- Close attention must be paid to the concrete consistency to ensure the finish tines or intrusions are consistent with the requirements of the specifications.
- Ensure the finishing tools (tine, burlap, broom, etc.) are properly adjusted and kept clean.
- Each tine should be a rectangular shape and the width should be between 3/32” and 1/8".
- The tining should produce a negative intrusion into the surface and not produce any positive texture.
- Ensure the tining spaces are evenly spaced at 3/4".
- Ensure the depth of the tining is between 3/32" and 1/8". (The distance from the edge of a quarter to the top of President Washington's head is about 1/8".)
- Figure No. 3 is an example of a PCC bridge deck using the *longitudinal tining* option.
- Figure No. 4 is an example of a tining tool used for the *longitudinal tining* option.
- Grinding for smoothness, when necessary per Standard Specification Section 51-1.17, *Finishing Bridge Decks*, is still required.

Attention to all the details associated with placing the bridge deck concrete will result in a higher quality bridge deck that is quieter.

Special Situations

A bridge may be in a noise sensitive area if it is adjacent to a residential area, hospital, school, park, or hotel. For projects within a noise sensitive area, defined by the Division of Pavement, Pavement Policy Bulletin, PPB 09-02, *Quieter Pavement Strategies for Noise Sensitive Areas*, and the SSP *Bridge Deck Surface Texture*, the district may request that only the *longitudinal grinding and grooving* option be specified. If a bridge project is in a noise sensitive area and the SSP *Bridge Deck Surface Texture* is not in place, or to address any other quiet bridge deck questions or concerns, contact OSC Headquarters for assistance. PPB 09-02 is available at the following intranet link:

http://onramp.dot.ca.gov/hq/maint/pavement/Quieter_Pavements.htm

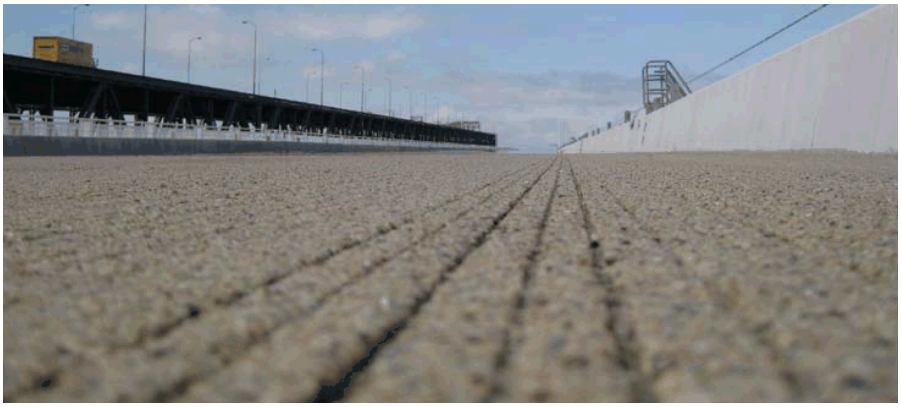

Figure No. 1: Example of Longitudinally Tined Polyester Concrete Bridge Deck Surface.

Figure No. 2: Example of a PCC Bridge Deck Using the "*Longitudinal Grinding and Grooving*" Option.

Figure No. 3: Example of a PCC Bridge Deck Using the *Longitudinal Tining* Option.

Figure No. 4: Example of the Tining Tool used for the *Longitudinal Tining*

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115-1.0	03/01/1998	Inspection of Electrical, Mechanical, Water and Wastewater Work

FOR RALPH P. SOMMARIVA, Chief
Office of Structure Construction

*Denotes the document is a Bridge Construction Bulletin

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Inspection of Electrical, Mechanical, Water and Wastewater Work

It is the Structure Representative's responsibility to ensure that all electrical, mechanical, water and wastewater (EMW&W) work complies with the plans and specifications. However, EMW&W work is a specialized and complex type of work with which the Structure Representative may not be totally familiar.

In order to be assured that the EMW&W work complies with the plans and specifications, the Structure Representative is encouraged to contact the Electrical, Mechanical, Water and Wastewater Branch (Telephone 916-227-8337 calnet 498-8337) to keep them informed of the work progress, to discuss any problems encountered, and to make arrangements to have EMW&W personnel make periodic inspections of the work as it progresses towards completion.

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MEMO NO.	ISSUE DATE	TITLE
120-1.0	08/30/2013	Submitting Falsework Shop Drawings
120-2.0	08/30/2013	Impaired Clearances at Falsework Traffic Openings
120-2.1	05/20/2010	Reporting of Impaired Clearances at Falsework Traffic Openings
120-3.0	05/30/1995	Falsework Shear Values for Bolts/Anchors in Concrete
120-4.0	06/13/2003	Reviewing and Submitting Temporary Support (Guying Plans) & Working Drawings
120-5.0	04/30/2008	Field Review of Tempoary Structures (Falsework)
120-6.0	04/30/2008	Contractor Standard Falsework Details

ROBERT A. STOTT
Deputy Division Chief
Structure Construction
Division of Engineering Services

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Submitting Falsework Shop Drawings

Falsework Drawings
Directions for submittal of falsework shop drawings to the Structure Construction Falsework Specialist are included in the *Falsework Manual*, Chapter 2, *Review of Falsework Drawings*[1].

Falsework Specialist
Structure Construction Falsework Specialist has the following responsibilities pertaining to falsework:

- Participate in falsework research projects and provide input relative to changes in specifications and policy.
- Perform a cursory review of all falsework shop drawings and calculations for falsework adjacent to or over railroads and forward the drawings and calculations to the railroad for their review and acceptance. The Falsework Specialist is the liaison between the project and the railroad.
- *Spot check* falsework shop drawings and calculations for falsework not adjacent to or over railroads. The drawings to be *spot checked* are selected at random with the objective of ascertaining compliance with current falsework directives and policies.
- Act as consultant to the Structure Representative and provide guidance with complicated falsework problems and resolve questions involving falsework policy.
- Act as liaison between Structure Design and the Structure Representative to review all drawings and calculations for contractor-designed temporary bridges or other temporary facilities which cross a traveled way being used by public traffic.

Attachment No. 1 contains an example of an authorization letter that may be used as guidance when authorizing a falsework submittal. This must be accompanied by a *Temporary Structure Plan Analysis Report*. See *Falsework Manual*, Chapter 2, *Review of Falsework Drawings*, and Bridge Construction Memo 3-6.0, *Shop Drawing Review for Temporary Structures*.

[1] http://dschq.dot.ca.gov/sc_manuals/falsework/Chapter_2.pdf

SAMPLE
FALSEWORK SHOP DRAWING AUTHORIZATION LETTER

DEPARTMENT OF TRANSPORTATION
<Your Office Address>
<Your Office Phone>

Flex your power!
Be energy efficient!

Date: <Date>

File: <Project Name>
<Co/Rte./Pm>
<Job EA>

<Contractor Name>
<Contractor Address>

Dear : <Responsible Person>,

The falsework shop drawings for Camarillo Overhead and Separation (Widen), Bridge No. 52-16, as revised December 1, 2012, have been reviewed and are authorized to the extent provided in the Standard Specifications, Section 5-1.23.

Your attention is directed to your responsibilities pursuant to the Standard Specifications, Sections 5-1.23, 7-1.04, and 48-2.0, as well as the Construction Safety Orders.

You are reminded that the falsework must be constructed to conform to the falsework drawings, that the materials used must be of the quality necessary to sustain the stresses required by the falsework design, and that the workmanship must be of such quality that the falsework will support the loads imposed on it without excessive settlement or take up beyond that shown on the falsework drawings.

Sincerely,

Resident Engineer

c:

Enclosure:

"Caltrans improves mobility across California"

Impaired Clearances at Falsework Traffic Openings

To ensure the movement of goods across the State Highway System (SHS) it is necessary to provide traffic openings through construction projects. During bridge construction, openings through the falsework allow the passage of traffic to meet this requirement. When these traffic openings through the falsework restrict the height of vehicles using the SHS, it is necessary to notify the Division of Traffic Operations, Office of Truck Services. The *Highway Design Manual* mandates that traffic openings should have a vertical clearance of at least 15 feet and clearances less than 15 feet require a design exemption. The timely notification of temporary impaired clearances due to falsework is essential to the safe routing of oversize vehicles and ensuring the safety of the traveling public, the contractor's personnel and Caltrans staff.

The Structure Representative shall submit written notice to the Resident Engineer when falsework will temporarily impair clearance on the State highway or roadway. This notification is accomplished by completing form TR-0029 or TR-0019, *Notice of Change in Clearance or Bridge Weight Rating*. Time requirements for notification of impaired clearance are found in the Standard Specifications, Section 7-1.04[1], *Public Safety*, and are usually no less than 20 and not more than 90 days prior to the impairment[2]. The *Construction Manual* still requires that the notification to the Transportation Permits Branch be within 15 days, the 5-day difference is for the Structure Representative to complete the calculations and give them to the Resident Engineer.

The detailed procedures for notification of temporary impaired clearances are contained in the:

- *Bridge Construction Records and Procedures Manual*, Bridge Construction Memo 2-20.0, *Notice of Change in Structure Clearance or Permit Ratings.*
- *Construction Manual*, Section 3-703A, *Temporary Clearance and Bridge Permit Rating Changes*.

The notification to the Resident Engineer should give anticipated dimensions of the impaired opening. The anticipated clearance should be calculated from the contractor's falsework submittal and verified using actual field dimensions. Structure Representatives, when performing the clearance calculations, should include an allowance for: falsework stringer deflection, adjustment of falsework grades, changes in pavement elevations, settlement, etc. The attached form SC-12.6.1, *Report of Falsework Clearance,* provides a methodology for determining the clearance under falsework at traffic openings. It should be filed in Category 12.6, *Falsework Plan,* of the project files.

[1] 2006 Standard Specifications, Section 7-1.09, *Public Safety*.
[2] Review the Contract Special Provisions for amendments or changes to these requirements.

The Structure Representative, immediately after the clearance is impaired, shall verify the dimensions of the impaired opening and notify the Resident Engineer of any necessary revisions to the clearance ensuring that allowances for settlement, stringer deflection, etc. continue to be taken into account. If the clearance is less, the Structure Representative should consider halting operations and removing the stringers already set, until clearance issues are resolved. The instructions within this section apply to any falsework being set over a traveled way, even if the resulting clearance satisfies legal height or load limitations. When the temporary impaired clearance is removed, the Structure Representative shall give the Resident Engineer written notice of the restored or revised clearance.

When falsework will not be removed by the tentative end date, an update within 15 days of the tentative end date previously submitted, needs to be made to the Transportation Permits Branch.

Structure Representative Responsibilities

- Determines the theoretical clearance of the falsework traffic opening.
- Verifies that the opening clearance is greater than or equal to the dimensions for traffic openings given in the Contract Special Provisions.
- Completes the form TR-0019 or TR-0029, *Notice of Change in Clearance or Bridge Weight Rating*.
- Reports the impaired clearance to the Resident Engineer or Transportation Permits Branch or both, depending on District protocols, 15 days prior to erecting falsework.
- Verifies Transportation Permits Branch received the *Notice of Change in Clearance or Bridge Weight Rating*, (return fax).
- Measures the clearance when the impairment is placed.
- Verifies that the measured clearance is greater or equal to the clearance previously reported.
- Takes appropriate action if the measured clearance is less than that previously reported.
- Submits a revised clearance if it is different or if the ultimate clearance will be less than previously reported.
- Submits a revised clearance prior to falsework being lowered if the clearance will be less than that previously reported.
- Attach a copy of the Contract Special Provision page showing the traffic opening requirements for the structure under consideration to the file copy.
- If applicable attach copies of Contract Change Orders that modify the dimensions of the traffic opening listed on the *Notice of Change in Clearance or Bridge Weight Rating* to the file copy.
- Have the Bridge Construction Engineer review and initial the *Notice of Change in Clearance or Bridge Weight Rating*.

Bridge Construction Engineer Responsibilities

- Ensures that the determination of the theoretical clearance is correct.
- Ensures the Contract Special Provision requirements have been met.

- Reviews the *Notice of Change in Clearance or Bridge Weight Rating* and if it is correct, *initials* the *Notice of Change in Clearance or Bridge Weight Rating*.

Structure Construction Oversight Engineer Responsibilities

- With Local Agency Structure Representative:
  - Ensures that the determination of the theoretical clearance is correct.
  - Ensures the Contract Special Provision requirements have been met.
  - Reviews the *Notice of Change in Clearance or Bridge Weight Rating*, and if it is correct, *initials* the *Notice of Change in Clearance or Bridge Weight Rating*.

Department of Transportation

REPORT OF FALSEWORK CLEARANCE

Form No. SC-12.6.1 (New 04/08) Formerly DS-OS C108

Job Stamp:

Date:

Bridge name:

Br. No. :

Co/Rte/PM:

Direction of travel:

Determination of falsework clearance:

a) Calculated or Measured Minimum vertical clearance: _____

Allowances:

b) Pavement elevation changes (- or 0) _____

c) Adjustment of Falsework grades (- or 0) _____

d) Falsework settlement (-) _____

e) Falsework stringer deflection (-) _____

f) Release of sand jacks (wedging) (-) _____

g) Calculated ultimate actual clearance[1]

h) Clearance to report[2]

[1] This value must be greater than that given in the Special Provisions.

[2] Calculated ultimate actual clearance rounded down to the nearest 3”

The clear horizontal opening is *(insert dimension between k-rail)* feet wide.

Remarks:

Instructions for Determination of Falsework Clearance

Use this form as an aid in determining the clearance at falsework openings. Reference BCM 120-2.0.

a) Prior to falsework erection this value is calculated by subtracting the falsework depth (soffit plywood, joist, nailers, and stringer) below the bridge soffit from the difference in elevation between the bridge soffit and roadway.

   After falsework erection this value is the measured distance between the roadway and the lowest edge of the falsework (generally the bottom flange of the stringer).

b) If there are plans to pave the roadway under the structure prior to removal of the falsework, the net thickness of the overlay will need to be subtracted from the clearance. The net thickness is used to account for any grinding that may take place prior to the placement of the final surfacing.

c) If the falsework is adjusted upwards a value of zero can be used to provide a slight buffer to the clearance.

d) The probable or anticipated settlement of the falsework.

e) Although the stringer deflection is generally compensated by the use of camber strips, the stringer itself will still deflect.

f) If traffic will be allowed under the structure between the time sand jacks (wedging) is blown (removed) and stringers are removed, this allowance needs to be included.

g) This is equal to the value of: value *a)* minus the summation of values *b)* through *f)*.

h) This is the value of *g)* rounded down to the nearest 3″, i.e. 16'-5.75″ would become 16'-3″ and 16'- 1″ would become 16' - 0″.

This is the value that should be used in form TR-0029, *Notice of Change in Clearance or Bridge Weight Rating*, when reporting to the Resident Engineer.

OFFICES OF STRUCTURE CONSTRUCTION *Bridge Construction Records and Procedures Manual*	10-02
BRIDGE CONSTRUCTION BULLETIN **Approved:** _____ **ROBERT A. STOTT** **Deputy Division Chief** **Offices of Structure Construction**	**File: BCM 120-2.1** **FALSEWORK** **Date: May 20, 2010** **Expires: July 1, 2012** **Supersedes:**

Subject: Reporting of Impaired Clearances at Falsework Traffic Openings

Background

To ensure the movement of goods across the State Highway System (SHS) it is necessary to provide traffic openings through construction projects. During bridge construction, openings through the falsework allow the passage of traffic to meet this requirement. When these traffic openings through the falsework restrict the height of vehicles using the SHS, it is necessary to notify the Division of Traffic Operations, Office of Truck Services. The Highway Design Manual mandates that traffic openings should have a vertical clearance of at least 15 feet and clearances less than 15 feet require a design exemption.

Recent events including field changes of the vertical clearance to less than 15 feet and incorrectly reporting the actual vertical clearance have required the Offices of Structure Construction to implement a quality assurance step into the process of reporting changes to vertical and horizontal clearances. The quality assurance review will require the Bridge Construction Engineer to review the "Notice of Change in Clearance or Bridge Weight Rating" before initial submission to the Resident Engineer or Truck Services.

Current Practice

Structure Representative:

- Determines the theoretical clearance of the falsework traffic opening using the guidance in BCM 120-2.0
- Verifies that the opening clearance is greater than or equal to the dimensions for traffic openings given in the Contract Special Provisions.
- Completes the form Form TR-0019 or TR-0029.
- Reports the impaired clearance to the Resident Engineer or Office of Truck Services or both depending on District protocols 15 days prior to erecting falsework.
- Verifies Office of Truck Services received the "Notice of Change in Clearance or Bridge Weight Rating." (return fax).

- Measures the clearance when the impairment is placed.
- Verifies that the measured clearance is greater or equal to the clearance previously reported.
- Takes appropriate action if the measured clearance is less than that previously reported.
- Submits a revised clearance if it is different or if the ultimate clearance will be less than previously reported.
- Submits a revised clearance prior to falsework being lowered if the clearance will be less than that previously reported.

New Practice

In addition to the above procedures the following additional duties have been added to the roles of:
Structure Representative:

- Attach a copy of the Contract Special Provision page showing the traffic opening requirements for the structure under consideration to the file copy.
- If applicable attach copies of Contract Change Orders that modify the dimensions of the traffic opening listed on the "Notice of Change in Clearance or Bridge Weight Rating" to the file copy.
- Have the Bridge Construction Engineer review and initial the "Notice of Change in Clearance or Bridge Weight Rating".

Bridge Construction Engineer:

- Ensures that the determination of the theoretical clearance is correct.
- Ensures the Contract Special Provision requirements have been met.
- Reviews the "Notice of Change in Clearance or Bridge Weight Rating" and if it is correct **initials the** "Notice of Change in Clearance or Bridge Weight Rating."
- Ensures remaining steps of the process are understood.

OSC Oversight Engineer

- With Local Agency Structure Representative:
    - Ensures that the determination of the theoretical clearance is correct.
    - Ensures the Contract Special Provision requirements have been met.
    - Reviews the "Notice of Change in Clearance or Bridge Weight Rating" and if it is correct **initials the** "Notice of Change in Clearance or Bridge Weight Rating."
    - Ensures remaining steps of the process are understood.

Related Guidance

Bridge Construction Records and Procedures, BCM 120-2.0, Impaired Clearances at Falsework Traffic Openings

Bridge Construction Records and Procedures, BCM 2-20.0, Notice of Change in Structure Clearance or Permit Rating
Construction Manual, Section 3-705, Public Safety
Standard Specifications, 7-1.09, Public Safety
Contract Special Provisions, Maintaining Traffic and/or Order of Work
Highway Design Manual, Section 204.8, Grade Line of Structures

Falsework Shear Values for Bolts/Anchors in Concrete

General Information

It is extremely important to consider the effect on concrete of bolts loaded in shear parallel to the face of the concrete especially within 6 inches of a concrete edge. Examples of these loading conditions are bolts inserted in the edge of decks which may support vertical falsework loads, or for pins which may be placed in decks and loaded horizontally as shear resistance for K-Rail.

Bar reinforcing steel under, and perpendicular to, the axis of a bolt imbedded in an edge of deck provides some resistance to shear failure of the concrete. Bar reinforcing steel under, but parallel to the bolt axis (without perpendicular reinforcing), provides little additional resistance to concrete shear failure. Unreinforced concrete will offer the least resistance to concrete shear failure.

Bolts or anchors inserted into concrete with impact drills or hammers will generally exhibit lower shear resisting capacity because of potential fracturing of the concrete. The lowest shear resisting capacity may well be furnished by wedge fit type anchors in holes made by impact tools since the wedging action can induce additional fracturing of the concrete.

Research

Bolts of 60 ksi tensile strength, approximately 5/8" in diameter and approximately 11-7/8" long, were cast about 7-7/8" deep at various edge distances (d) up to about 6 inches in unreinforced concrete blocks. The bolts were load tested at right angles to the block surfaces after the concrete had gained sufficient strength. Published test results of concrete failure due to the lateral loads on the bolts showed that the modes of failure consisted of concrete failure with and without wedge cones, or with pullout cones, or by shear failure of the bolts.

Test results for concrete shear strengths adjusted to an averaged concrete compressive strength f_c' of approximately 3,000 psi have been plotted in Figure 1. Assuming[2] that the shear strength of concrete is a function of f_c' a family of curves related to the 3,000 psi curve were generated and included in Figure 1. The curves of Figure 1 represent ultimate values which need to be adjusted with an appropriate safety factor which is determined from Table 1.

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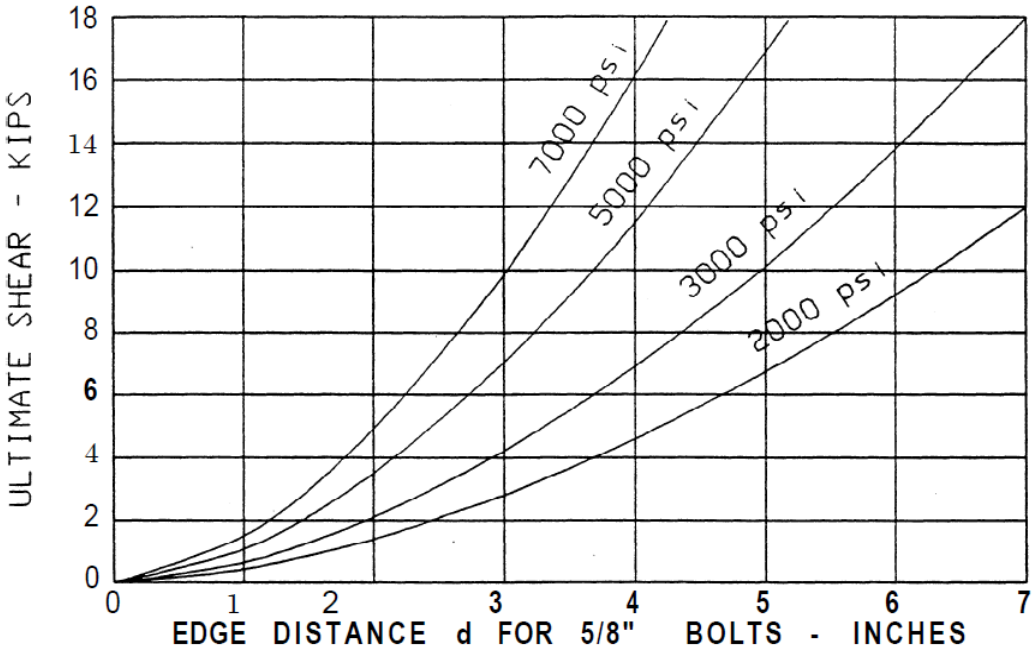


FIGURE 1

One of the most common usages of embedded bolts is depicted in Figure 2. Figure 2 depicts one direction of loading with respect to the concrete surface and shows dimension d which represents the distance from the edge of the concrete to the center line of the bolt.

Permitted Use

Determine the distance the loaded bolt will be from the loaded edge 1 or change in section, of the concrete. The term “bolt” shall be meant to include inserts, rods, or other similar devices. Haunched concrete sections similar to the underside of bridge deck overhang sections, should not be given additional value. Select a concrete shear strength related to an appropriate ultimate concrete strength curve from Figure 1. Divide the selected ultimate concrete shear strength value by an appropriate safety factor as determined from Table 1.

Table 1 lists minimum safety factor values to be used for cast-in- place bolts. A minimum safety factor of 2 may be used where reinforcing is located normal to the axis of the bolt in the concrete shear loaded zone provided the reinforcing is located between the concrete face and the midpoint of the embedded bolt length. This reinforcing must extend through the concrete failure zone (See Figure 2) to sound concrete. Otherwise use a safety factor of 2.25 or higher.

A minimum safety factor of 2.75 may be used when reinforcing steel located parallel to the bolt axis (without reinforcing normal to the bolt axis) will be within the shear loaded zone and will have a length reaching to at least the midpoint of the embedded bolt. If no parallel reinforcing will be within a shear loaded zone use the higher safety factor of 3.0.

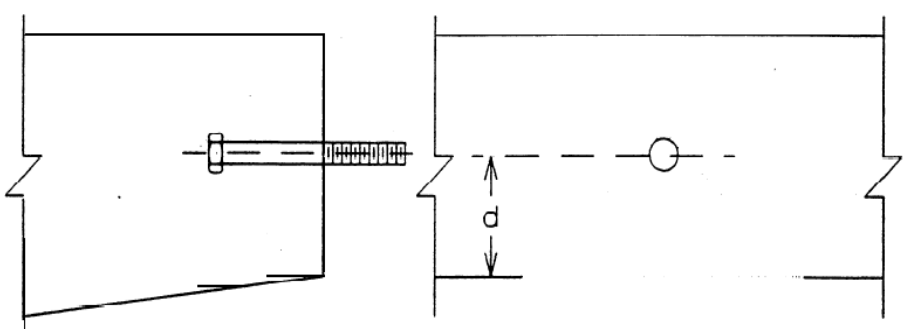
Safety Factors:		
Reinforcing Type	Cast – In – Place Bolt	Bolt Insert
Bar (s) within 2″ of the concrete face located normal to the bolt axis on the loaded side of the bolt.	2.0 – 2.5	2.25 – 2.50
Bars parallel to the bolt axis on the loaded side of the bolt (no normal reinforcing).	2.75 – 3.0	2.75 – 3.0
No reinforcing on the loaded side of the bolt.	3.0	3.0
Torque tightened bolts, regardless of location of reinforcing	3.0	3.0

Table 1

Shear loaded zones are depicted in Figure 2 and are described as follows:

Cone Pullout - A cone with the apex situated behind the embedded end of the bolt with the center line of the cone on the axis of the bolt and with sides sloping a minimum of 45° away from the bolt axis.

Trapezoidal Wedge - A trapezoid volume with the base on the concrete surface a distance d from the bolt center line, having sides sloping inward at 45° angles meeting a plane of length equal to the bolt diameter at the opposite side of the bolt, and with this area having a volumetric length equal to the length of the embedded portion of the bolt.



Cone Pullout Trapezoidal Wedge

FIGURE 2

When information about reinforcing on the loaded side of a bolt cannot be verified the highest safety factor will be used. No increase or reduction in the safety factor value, will be given to bolts with embedded bolt heads.

Figure 3 illustrates a single bolt placed in a deck overhang. Bolts shall not be spaced closer than 8 times the d dimension or 3.5 times the sum of the embedment lengths of adjacent bolts whichever is larger. The 8 times the d dimension is a recommendation made in the published test results. The 3.5 (two times the tangent of 60°) times the sum of the embedment lengths is based on evidence that the concrete can fracture or fail in shear at an angle of 60° to the axis of the bolt.

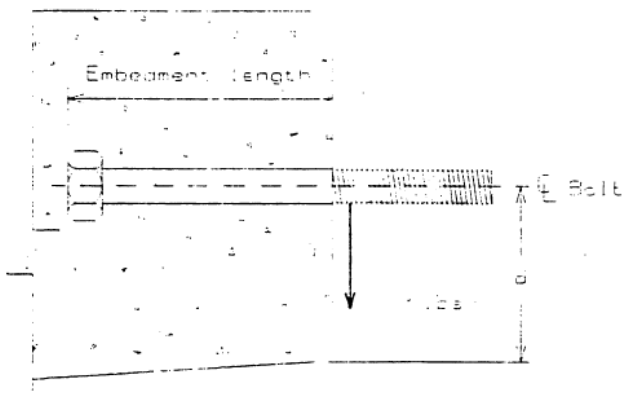


FIGURE 3

Since one of four types of failure modes occurred during testing no adjustments need be made for various bolt sizes up to 5/8" in diameter. For bolts larger than 5/8" diameter use dimension d as being equal to the distance from the edge of the concrete to the nearest portion of the bolt hole (In Figure 3 this dimension would be d minus one-half the bolt diameter).

It is always a good idea to test load a typical section of falsework supported by bolts or inserts to twice the anticipated loading at a safe location.

<u>Example:</u>

Assume 5,000 psi concrete in a deck overhang with no bottom mat reinforcing for which holes for 5/8" bolts are to be drilled 3.75 inches from the soffit with air tools for torque type bolt inserts that are to be used for supporting a falsework system for removal of concrete railing. Determine bolt capacity and minimum bolt spacing for 5 inches of maximum embedment.

From Figure 1 the ultimate value for shear failure may be selected as 10.1 Kips, and from Table 1 the safety factor value should be no less than 3.0.

Assumed safe working load for the bolt = 10.1/3.0 = 3.4 Kips. Minimum bolt spacing is the larger of 8(d) or 3.5(5" + 5"):

8(3.75)	=	30.0 Inches
3.5(5"+ 5")	=	35.0 Inches + This governs

References:

1. Ueda,T.; Stitmannaithum,B.; and Matupayont,S., "Experimental Investigation on Shear Strength of Bolt Anchorage Group," ACI Structural Journal, V. 88, No. 3, May-June 1991, pp.2920300.

2. ACI Committee 349, "Proposed Addition to: Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-76)"and "Addition to Commentary on Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-76)," ACI Journal, *Proceedings* V. 75, No. 8, Aug. 1978, pp. 329-347,

Additional Information

Consult the *Bridge Construction Records and Procedures Manual* Memo 135-5.0 and Memo 168-2.0 regarding bolts in concrete. Additional information on concrete anchorage devices may be found in Section 75-1.03 of the Standard Specifications. While these references pertain to permanent installations, the guidelines are worthy of note. Installation for temporary work do not require Translab approval. It is important however, that manufacturer's recommendations be followed except where there will be obvious deviations from this memo.

Volume II

REVIEWING AND SUBMITTING TEMPORARY SUPPORT (GUYING PLANS) & WORKING DRAWINGS

Temporary support (guying) plans submitted by the Contractor must be reviewed by the Offices of Structure Construction staff thoroughly and independently. You may find the following references useful during this review:

- Standard Specifications:
  Section 52-1.07, 'Placing'
  Section 7-1.09, 'Public Safety'

- Falsework Manual:
  Section 4-5, 'Cable Bracing Systems'
  Falsework Memo No. 9, 'Short Poured-In-Place Concrete Piles to Resist Uplift and Lateral Loads'.

- Contract specific Special Provisions

The minimum horizontal load to be allowed for wind on steel cages shall be the sum of the products of the wind impact area and the applicable wind pressure value for each height zone. The wind impact area is the total projected area of the cage. Wind pressure values shall be determined from the following table:

HEIGHT ZONE (METERS ABOVE GROUND)	WIND PRESSURE VALUE (Pa)
0-9	960
9-15	1200
15-30	1440
Over 30	1675

Temporary support plans shall include sufficient detail to ensure stability of reinforcing cages during all phases of construction. The plan shall also include provisions for keeping stable the column cage and forms during the transition from one stage to the next as well as a list of all equipment to be utilized to handle the erection.

Copies of working drawings and design calculations per Section 52-1.07, 'Placing' of the Standard Specifications shall be submitted to the Offices of Structure Construction HQ in Sacramento after approval by the Structure Representative.

When such plans as described by the previous paragraph are adjacent to railroads, the procedure for approving these plans will be in accordance with the requirements for submitting falsework when railroad companies are involved. A complete explanation of this procedure may be found in the Falsework Manual Section 2-1.06B, 'Procedure when Railroad Company Approval is Required'.

Volume II

FIELD REVIEW OF TEMPORARY STRUCTURES (FALSEWORK)

Temporary structures erected by contractors in connection with the construction of state highway structures need to remain safe, stable, and serviceable throughout their design life. Failure of these temporary structures (falsework) could be catastrophic. The OSC practice is to use all contract requirements, experience, and 'best engineering practice' to prevent falsework failures.

OSC staff will have the following responsibilities as a minimum to ensure a safely constructed system:

Structure Representative / Assistant Structure Representative

- Ensure that falsework plan(s), materials incorporated, and construction methods meet contract requirements and the best general practices represented in the Falsework Manual.
- Ensure that falsework is designed for the intended loads that may act upon it.
- Perform an independent stress analysis of the contractor's submitted falsework system.
- Ensure that all pertinent load tests are performed and properly documented.
- Ensure that falsework is constructed as per the approved plan.
- Ensure that the contractor's falsework plan demonstrates stability during all phases of erection and removal.
- Ensure that the falsework plan and construction meets all California Department of Occupational Safety and Health (Cal OSHA) applicable safety orders.
- Ensure that the falsework is inspected and certified by the contractor's falsework designer or his authorized representative pursuant to the requirements in Article 1717 of the Construction Safety Orders, the Caltrans Standard Specifications, and the contract special provisions.
- Ensure that jacking and displacement monitoring systems are approved and in place prior to jacking.
- Work closely with the contractor's falsework foreman to coordinate all aspects of erecting, grading, and removing the falsework safely.

OSC Oversight Engineer

- With Local Agency Structure Representative, perform a field review of falsework installations prior to placing or removing concrete, or jacking of temporary support systems. (See BCM 2-19.0, "Administration of Special Funded Projects.")

Bridge Construction Engineers

- With Structure Representatives perform a field review of falsework installations prior to placing or removing concrete, or jacking of temporary support system.

Area Construction Managers

- Periodically perform a field review of falsework installations with Bridge Construction Engineers in their areas.

Volume II

CONTRACTOR STANDARD FALSEWORK DETAILS

Contractors typically use standard falsework details to supplement their project specific falsework plans. As an attempt to accelerate bridge construction Contractors have submitted their standard falsework details to Sacramento for approval.

The Sacramento office of Structure Construction has reviewed some of these details and has placed the reviewed Contractor's Standard Falsework Details, calculations, and the State's review calculations on the OSC Intranet site to facilitate a Structure Representative's review of a contractor's standard falsework details.

Considering the wide and varied usage of these details by contractors the Sacramento office of Structure Construction is offering the following practice for use by Structure Representatives when reviewing falsework with standard falsework details:

- Access the OSC intranet site
- Locate the Falsework Technical Team (intranet) Page
- Find the Contractor's Standard Details Link
- Find the detail indicated for use
- Review the detail and related calculations to ensure it matches your submittal and intended use.
- The limitation of use for the detail is shown, if your intended use is different, you may need to perform the check calculation yourself.
- Details on the standard falsework drawings should be listed on the falsework plans to ensure that only those details indicated to be used are reviewed and approved as part of the plan.

Use of a detail from a falsework standard details that is available on the website still requires the contractor to submit the standard details and supporting design calculations to the Engineer along with working drawings and calculations for falsework proposed for use at bridges.

The use of the standard falsework details available on the OSC Intranet are intended solely for the Structure Representative's use in checking falsework submitted by the contractor/owner of the standard falsework details. Use of a checked standard detail from the intranet for a set of plans submitted by another contractor is ethically improper and is not to occur.

The proper use of these standard falsework details and associated calculations requires a thorough understanding of the principles of civil engineering design, familiarity with falsework specifications and the use of sound engineering judgment.

Process for submission and review of Contractor Standard Falsework Details:

Contractor submits 6 sets of Standard Falsework Details and 3 sets of supporting calculations to:

Falsework Engineer
Offices of Structure Construction
Mail Station 9-2/11H
1801 30th Street
Sacramento, CA 95816

Falsework Engineer will perform an independent review of the Standard Falsework Details
This is an iterative process with the Contractor to determine "limits of use" for the individual details within the drawings.

Upon approval of the Standard Falsework Details, the drawings and supporting calculations (Contractor and State) will be posted on the OSC Intranet site.

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BRIDGE CONSTRUCTION MEMO 122-0.0

SHORING

June 1, 1997

Sheet 1 of 1

Volume II

TABLE OF CONTENTS FOR SECTION No. 122

Memo No.	Date	Title
122-1.0	12-05-89	SUBMITTING SHORING PLANS

RALPH P. SOMMARIVA, Chief
Office of Structure Construction

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BRIDGE CONSTRUCTION MEMO 122-1.0

SHORING

June 1, 1997

Volume II

SUBMITTING SHORING PLANS

General Information

Review of shoring plans shall be in accordance with the guidelines and policy set forth in the Office of Structure Construction Trenching and Shoring Manual.

Shoring Which is Not Adjacent to or Under a Railroad

When the Structure Representative has completed reviewing the shoring plans and calculations, and has determined that the shoring plans meet specification requirements, a letter shall be sent to the Contractor stating that the drawings have been reviewed and approved. This letter authorizes the Contractor to begin constructing shoring. Note that the Contractor must not begin shoring construction until such time as this letter has been issued to the Contractor. The letter should state that the approval is to the extend provided under Section 5-1.02 of the Standard Specifications. This letter should also direct the Contractor's attention to his/her responsibilities under Sections 5-1.02 of the Standard Specifications. It should remind the Contractor of the requirement that the shoring must conform to the shoring plans, that the materials used must be of the quality necessary to sustain the stresses required by the shoring design, and that the workmanship shall be of such quality that the shoring will support the loads imposed on it without excessive movement (See Attachment No. 1 for an example of a properly prepared shoring approval letter.)

For shoring that is not adjacent to or under a railroad, it is mandatory that one copy of the shoring plans, and one copy of the Structure Representatives calculations be submitted to the Sacramento Office of Structure Construction, immediately after they have been reviewed and approved by the Structure Representative.

A letter of transmittal must accompany all shoring plans and calculations that are submitted to the Office of Structure Construction Falsework Review Unit. Include a copy of the approval letter that was issued to the Contractor.

The Structure Representative should retain one set of the approved shoring plans and calculations in the job files.

Shoring Adjacent to or Under Railroads

When shoring is adjacent to or crosses under a railroad, additional requirements must be complied with. Any shoring within 15' of the centerline of tracks is considered adjacent.

The Structure Representative will check the shoring plans, and if necessary, return them to the Contractor for correction. After shoring plans and calculations have been reviewed by the Structure Representative and he/she is satisfied that the shoring plans meet specifications, the following items are to be sent to the Office of Structure Construction.

(1) Five copies of Contractor's shoring plans if the Union Pacific (Southern Pacific), Burlington Northern Sante Fe or Western Pacific railroad are involved.

(2) Three copies of Manufacturers' data relative to manufactured devices.

(3) Three copies of Contractor's calculations.

(4) Three copies of the Structure Representative's calculations.

Note: One copy of the above is for the Sacramento Office of Structure Construction's use, and the other copies are forwarded to the railroad by the Office of Structure Construction. In the event that railroad personnel at the job site need copies of the above information, they are to obtain it from their headquarters.

When the above noted data is submitted to the Office of Structure Construction, it shall be accompanied by a letter of transmittal in which the Structure Representative will list the information submitted, and state that the shoring plans and calculations have been reviewed and that they are considered to be satisfactory. The Office of Structure Construction will review this data. The Structure Representative should not stamp the shoring plans 'Approved' until they have been notified that the railroad has reviewed and accepted the shoring plans.

Incomplete or unsatisfactory data will be returned to the Structure Representative for correction. Upon confirming that the plans and calculations are complete and satisfactory, the information will be forwarded to the railroad for their review and acceptance.

When the Sacramento Office of Structure Construction and railroad reviews are complete, and the Sacramento Office of Structure Construction advises the Structure Representative that the railroad considers the shoring plans to be satisfactory, the Structure Representative will send a letter to the Contractor stating that the plans have been reviewed and approved. This letter authorizes the Contractor to begin constructing the shoring. Note that the Contractor must not begin shoring construction until such time as the approval letter has been issued to the Contractor. (See Attachment No. 1 for an example of a properly prepared shoring approval letter.)

Shoring Review Unit

A Shoring Review Unit will be operational in the Sacramento Office and this unit will perform the following functions:

1. Participate in shoring research projects and provide input relative to changes in specifications and policy.

2. Review all shoring plans and calculations for shoring adjacent to or under railroads and forward the plans and calculations to the railroad for their review and approval. The Shoring Review Unit provides the liaison between the job and the railroad.

3. "Spot Check" shoring plans and calculations for shoring not adjacent to or under railroads. The plans to be "spot checked" are selected at random with the objective of ascertaining that current shoring directives and policies are being complied with.

4. Act as consultant to the Structure Representative and provide guidance with complicated shoring problems and resolve questions involving shoring policy.

DEPARTMENT OF TRANSPORTATION
ENGINEERING SERVICE CENTER
OFFICE OF STRUCTURE CONSTRUCTION MS-9
P.O. Box 942874
SACRAMENTO, CA 94274-0001

(916) 227-7777
(916) 227-8835
FAX (916) 227-8179

June 1, 1997

Co-Rte-PM
00-000104

Sample Construction Co.
Sample Lane
Sample, CA 00000-0000

Gentlemen:

The shoring plans for the temporary retainment of soil and embankment material at Abutment 1 of General Avenue Overcrossing, Bridge No. 2B-X4X, dated August 14, 1989, have been reviewed and are approved to the extent provided in Section 5-1.02 of the Standard Specifications. [Add the following when using the contractor's/consultant's soil parameters; This approval is based upon the soil parameters submitted by your consultant.] [Add when needed for trench shields; Note that your submittal did not include calculations from the trench shield design engineer. It is therefore understood that your licensed engineer has verified that the shields themselves are capable of sustaining the loads allowed by the shield manufacturer.]

Your attention is directed to your responsibilities pursuant to Section 5-1.02 and 7-1.09, of the Standard Specifications as well as the Construction Safety Orders.

You are reminded that the shoring must be constructed to conform to the shoring drawings, that the materials used must be of the quality necessary to sustain the stresses required by the shoring design, and that the workmanship shall be of such quality that the shoring will support the loads imposed on it without excessive movement.

Sincerely,

Y.Y. Resneer
Resident Engineer

by

W.W. Strurep
Structure Representative

c:

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 122-2 SHORING**
Approved: Original Signed by Dolores Valls **Dolores Valls, Deputy Division Chief** **Offices of Structure Construction**	**Date: October 15, 2002** **Expires: July 1, 2003** **Supersedes: BCM 122-2 Dated May 19, 1997**

Subject: Support from Roadway Geotechnical Engineering

Shoring systems requiring the use of slope stability (i.e. excavation exceeding the Cal OSHA sloping or benching requirements) should be reviewed by the Roadway Geotechnical Engineering unit for adequacy. To obtain assistance from the Roadway Geotechnical Engineering unit, contact the appropriate Geotechnical Design Branch (North, South, West) that your project is in using the following link:

http://onramp.dot.ca.gov/hq/esc/gs/design.shtml

cc: BCR&P Manual Holders
R. Pieplow, HQ Const.
Consultant Firms
J. Van Velsor, Geotech.Services

BRIDGE CONSTRUCTION BULLETIN **Approved:**___Original signed by_____ **DOLORES VALLS, Deputy Division Chief** **Offices of Structure Construction**	**File: BCM 122-3** **SHORING** **Date: December 21, 2001** **Expires: None** **Supersedes: None**

Subject: Torvane Usage for Sloped Excavations

The Offices of Structure Construction has received and approved a contractor's standard shoring plan utilizing a Torvane (or shearvane) device. The Torvane is a soil-testing tool utilized for rapid determination of cohesive soils' undrained shear strength (Su). The use of Torvane is an acceptable practice according to CalOSHA for determining the maximum allowable slope of an excavation.

The following soil properties were assumed for the evaluation of such a standard plan. If your soil properties differ significantly, additional analysis will be required:

? = 120 pcf (1920 kg/m^3)
? = 25°

Projects utilizing a Torvane will additionally require the following conditions be met and addressed with each shoring submittal:

- Tension cracks must be prevented from developing at top of excavation.
- Water must be prevented from ponding at the top of and toe of the slope.
- Soil must be consistent and homogeneous throughout the excavation.
- CalOSHA requirements must be adhered to at all stages of the excavation.

In situations where the contractor is proposing to use a Torvane, please consult the Trenching and Shoring Senior Specialist in the HQ Offices of Structure Construction at (916) 227-8060 for assistance.

c: BCR&P Manual Holders
CMB - Consultant Firms
RPieplow, HQ Division of Construction

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BRIDGE CONSTRUCTION BULLETIN	**File: BCM 124-2**
	D E M O L I T I O N
Approved: R. P. SOMMARIVA, Chief Office of Structure Construction	**Date: January 2, 1998** **Expires: January 1, 1999** **Supersedes: None**

Subject: Demolition Plan Review

This bulletin has been developed to assist field personnel responsible for the administration and review of bridge demolition plans. It is intended to address the current field issues regarding review of demolition plans and the quality assurance measures to be employed when reviewing demolition operations. The review and processing of the demolition plan should be documented thoroughly.

One copy of the approved drawings, one copy of the engineer's calculations, and the Structure Representative's calculations are to be submitted to the Sacramento office of Structure Construction, along with a copy of the approval letter sent to the contractor.

During the review, if it becomes apparent additional assistance is required due to the complexity of the structural analysis; the responsible design unit (OSD, CCMB, EFPB) will provide assistance. Before requesting assistance, discuss the plan with the Bridge Construction Engineer (Senior Bridge. Engineer) responsible for your project to obtain agreement that assistance is required. Once agreement is attained, contact the Design Senior or Project Manager responsible for your project and request assistance. A written request may be required depending on the complexity of the analysis.

The attachments are intended to be a guideline and not an all-inclusive list for bridge removal review. Bridge removal is a very important item to inspect for safety concerns regarding the public and contract personnel. Safety always comes first and if something does not look appropriate, do not compromise safety.

If further assistance is needed, contact the appropriate Sacramento OSC HQ Staff Engineer or Senior assigned to your district.

Attachment

cc: BCR&P Manual Holders
Consultant Firms
BGauger, Construction Program Manager

BRIDGE DEMOLITION PLAN REVIEWS

1. Recommend the Contractor investigate the as-built conditions of the structure and determine if special conditions exist (column pins and deck hinge(s) that may need to be restrained during bridge removal operations).

   A. Designer should include as-built plans for the structure to be removed in the RE Pending file, or the Structure Representative should request these upon first review of the RE Pending file.
   B. The Contractor should obtain as-builts from the district in which the work is situated per Standard Specifications.

2. Inform the Contractor to submit a complete bridge removal/demolition plan, prepared by an engineer who is registered as a Civil Engineer in the State of California, at least ten days prior to commencing demolition unless the Special Provisions state otherwise. This plan should detail the procedures and sequence for removing portions of bridge(s), including all features necessary to remove the bridge(s) in a safe and controlled manner. The stability of the structure must be demonstrated by calculations at each stage of removal. (This section does not apply when bridge removal is only specified for bridge railing work and/or widening work that requires small amounts of bridge removal.)

3. Remind the contractor that any vertical shoring used to stabilize the structure during removal operations shall conform to the provisions set forth in SS 51-1.06 "Falsework" of the Standard Specifications (unless otherwise noted in the Special Provisions).

4. If the contractor is removing a portion of a bridge (e.g. barrier rail only, edge of deck for a widening), the removal operations shall be performed without damage to any portion of the structure that is to remain in place. In such cases, tools with a manufacturer's rated striking energy in excess of 1625 J (1,200 foot pounds) per blow shall not be used for breaking or removing concrete which is attached to or supported by the bridge. (SS 15. 4.02)

PRE-JOB MEETING

Discuss the following items at the pre-job meeting:

-Standard Specifications (SS)
-Special Provisions (SP)
-Cal-OSHA Construction Safety Orders
-Falsework Manual (if necessary)

The Standard Specifications requires the Contractor to submit a demolition plan at least ten days prior to the beginning of bridge removal work. However, due to the complexity of a structure, the Special Provision may allow additional time.

INITIAL REVIEW

Perform an initial review and check for contract compliance. The plan must be generally suitable for the site conditions to be encountered and should provide adequate information clearly outlining the removal procedure proposed by the Contractor. The following are some basic items to check before your formal review.

1. Review the details and proposed staging of the removal operation. The plan should show the methods and sequence of removal. (SS 15-4)

2. Verify that the bridge removal plan is prepared by an engineer who is registered as a Civil Engineer in the State of California.

3. Calculations for vertical shoring, restraining systems for columns and deck hinges should be included, if applicable.

4. The design calculations shall be adequate to demonstrate the stability of the structure during all stages of the removal operations.

5. A complete, Contractor's equipment list with attachments, dimensions, axle loadings and equipment weights should be shown along with proposed placement locations to check for possible structure overloads. (SS 7-1.02)

6. Timeline, milestones, and contingency plans need to be included when dealing with traffic or other potential safety problems, traffic delays, and construction windows.

7. Protection of existing utilities and non-highway facilities should be covered. (SS 7-1.11 "Preservation of Property").

8. The locations of temporary hand railing and barrier railing for bridge decks should be shown for staged bridge removal. This detail is necessary for the protection of the employees and the public.

FORMAL REVIEW:

1. The safe work areas for the contractor's personnel, and Caltrans personnel should be shown on the demolition plan. The protection of public traffic (vehicles and/or pedestrians) and private property must be covered thoroughly in the demolition plan, including the safe routing of public traffic during demolition operations.

2. The location of the demolition equipment and the method in which the equipment is used to remove the structure should be reviewed. This may assist you in evaluating the stability of the structure under live loads during bridge removal. This is extremely important when a portion of the structure is to remain standing and equipment is to stay on the structure.

3. A freely falling mass or a falling mass attached to a cable; rope or chain shall not be used above or within 9m (30 feet) horizontally of any area open to the public unless adequate protective shields are in place (SS 15.4.02), and/or unless otherwise noted in the Special Provisions

4. The impact and possible damage due to excessive demolition debris falling and collecting on roadway section, adjacent structure, supporting falsework, protective cover, or even structural elements supporting the structure being removed need to be investigated. Again, the equipment loads should be considered in combination with debris build up. If needed, debris shields shall be used to protect the surrounding area, structural elements, and falsework and shall be detailed and shown on the demolition plan. Also, ensure there is a provision on the plans for dust control.

5. A timeline should be noted for each operation if traffic control is involved. When working within a "time-window", progress should be monitored against planned progress milestones and a contingency plan should be agreed upon prior to the start of removal operations if work falls behind planned progress.

6. The bridge removal plan shall conform to the requirements in Section 5-1.02, "Plans and Working Drawings," of the Standard Specifications. The number of sets of drawings and design calculations and times for review for any bridge removal plans shall be the same as specified for falsework drawings in Section 51.l.O6A, "Falsework Design and Drawings," of the Standard Specifications. (Unless otherwise specified in the Special Provisions.)

7. **One copy of the approved drawings, the engineer's calculations, and the Structure Representative's calculations are to be submitted to the Sacramento office of Structure Construction, along with a copy of the approval letter sent to the contractor.**

OTHER AGENCY REQUIREMENTS

1. The Contractor shall conform to all local sound, water, noise, and air pollution control requirements per Standard Specification Section 7, Special Provisions, and/or applicable permits.

2. The Contractor must get a disposal permit for disposal of material outside the highway right of way per Standard Specification Section 7- 1.13.

3. A Cal-OSHA permit is required for the demolition of any structure greater than 10.9 m (36 feet) in height. (Cal-OSHA Article 2 Section 341, "Permit Requirements")

4. If railroad review of the demolition plan is required on your contract, make sure to inform the agency well in advance of the work, especially if the demolition work is a critical item on the project schedule. Information regarding working with Railroads can usually be found in the Special Provisions Section, "Relations with Railroad Company". Also, inform the Contractor at the pre-job meeting regarding the possible time delay by impromptu plan submittal and revisions. The procedure for approving these plans is identical to that used for falsework plan approval. The instructions can be found in the Falsework Manual Section 2-1.06B 'Procedure when Railroad Company Approval is Required.'

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 124-3 DEMOLITION**
Approved: original signed by R.P.Sommariva **R. P. SOMMARIVA, Chief** **Office of Structure Construction**	**Date: May 19, 1999** **Expires: May 19, 2000** **Supersedes: None**

Subject: Demolition Over and/or Adjacent to Union Pacific Railroad Company Tracks

Railroad Guidelines

To expedite the review process of demolition plans by the Union Pacific Railroad Company (UPRR), it is advisable that the drawings submitted by the contractor adhere to the requirements of Union Pacific. The latest railroad's requirements are titled *GUIDELINES FOR PREPARATION OF A BRIDGE DEMOLITION AND REMOVAL PLAN FOR STRUCTURES OVER RAILROAD.* Refer to Attachment No. 1 for a copy of this guideline.

The contract special provisions will list the clearance requirements measured from the centerline of the railroad tracks. If clearances are not included in your contract documents, refer to UPRR Std. Dwg. 0035, "Barriers and Clearances to be Provided at Highway, Street, and Pedestrian Overpasses" for minimum construction clearance requirements. Refer to Attachment No. 2 for a copy of this drawing. This drawing shows the latest UPRR clearance requirements and will be incorporated into future contracts.

Where there is a conflict between the contract specifications and the guidelines issued by the railroad, the contract specifications shall prevail.

Railroad Requirements

The UPRR has requested that drawings accompanying demolition plans be submitted on 11"x17" (279.4 mm x 431.8 mm) sized paper. Future special provisions will be revised to state that the drawings for the railroad should be on 11"x17" (279.4 mm x 431.8 mm) sized paper. Until this request becomes a specification requirement, you can request that the contractor submit the three sets of drawings

accompanying demolition plans for railroad review be on 11"x17" (279.4 mm x 431.8 mm) sized paper.

Some common requirements are often overlooked and have resulted in submittals being returned by the railroad. The demolition plans should state that all demolition will comply with the latest railroad demolition guidelines. The demolition plans should note how the contractor will gain access to the site, particularly if they must cross the railroad tracks. Track protection details are shown in the aforementioned UPRR guidelines, and details must be included on the demolition plans.

The demolition plans should note if there are any existing drainage ditches or access roads being affected by the contractor's operations. If there are no existing drainage facilities or access roads, the demolition drawings should note this fact. Keep in mind that personnel from the railroad who are unfamiliar with the site often review the demolition plans.

The above railroad requirements should be discussed at the pre-construction meeting with the contractor. It should also be stated that approval of demolition plans over and/or adjacent to UPRR tracks will be contingent upon UPRR approving the plans.

Distribution of Demolition Plans

The Structure Representative will check the demolition plans, and if necessary, return them to the Contractor for correction. After the demolition plans and calculations have been reviewed by the Structure Representative and he/she is satisfied that the demolition plans meet the specification requirements, the following items are to be sent to the Office of Structure Construction Headquarters (OSC HQ):

1. Four copies of Contractor's demolition plans (a minimum of three sets of 11x17 drawings for the railroad is preferred)

2. Three copies of the Contractor's calculations, tabbed to show key elements affecting the demolition over and adjacent to the railroad company's tracks

3. Three copies of Structure Representative's calculations, tabbed to show key elements affecting the demolition over and adjacent to the railroad company's tracks

4. Three copies of manufacturer's data relative to manufactured devices

Note: One copy of the above is for the OSC HQ office use, and the other copies are forwarded to the railroad. In the event that railroad personnel at the job site need copies of the above information, they are to obtain it from their headquarters.

In order to complete the demolition review within the contract time specified, the Structure Representative should expedite their review and forward the submittal to the OSC HQ (Attention: John Gillis) via overnight mail.

When the above noted data are submitted to the OSC HQ office, a letter of transmittal from the Structure Representative shall accompany them. The transmittal letter shall list the information submitted and state the demolition plans and calculations have been reviewed and that they are considered to be satisfactory. The Structure Representative should not stamp the demolition plans 'Approved' until OSC HQ has notified them that the railroad has reviewed and accepted the demolition plans.

Railroad Review and Approval

Incomplete or unsatisfactory data will be returned to the Structure Representative for correction. The OSC HQ will review this data. Upon confirming that the plans and calculations are complete and satisfactory, the information will be forwarded to the railroad via overnight mail for their review and acceptance.

Please note that all correspondence with the railroad regarding the status of submittals under their review should be directed to John Gillis. At the railroad's request, in no case should you contact the railroad directly.

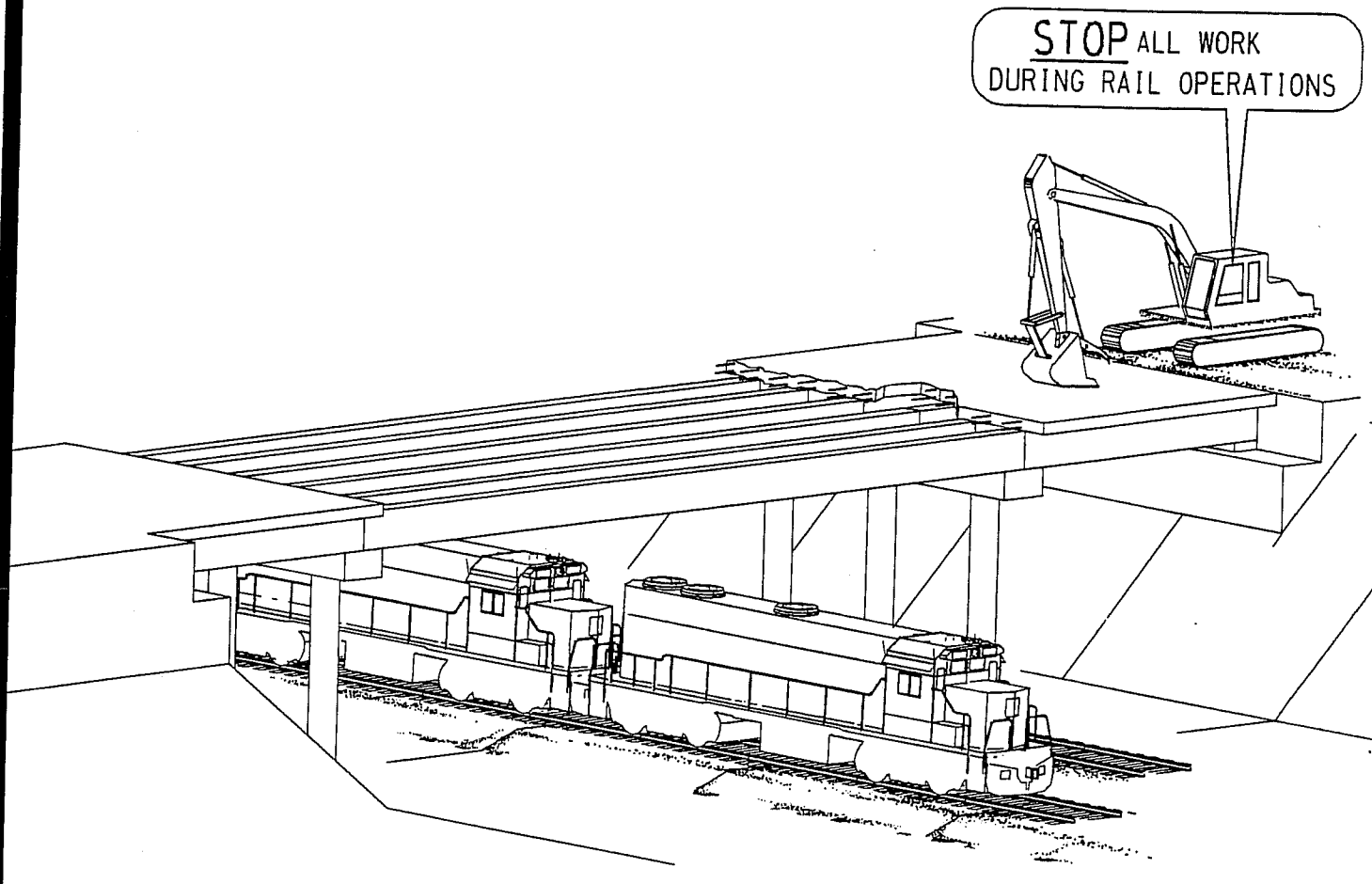
When the railroad completes their review and finds the plans to be acceptable, they will advise the OSC HQ who in turn will advise the Structure Representative that the railroad considers the demolition plans to be satisfactory. The Structure

Representative will then stamp the plans 'approved' and send a letter to the Contractor stating that the plans have been reviewed and approved. Assuming proper notification has been made to the UPRR that their horizontal and vertical clearances will be impaired and that a flagger is required, the Contractor may begin demolition work. Note that the Contractor shall not begin any demolition within the railroad right-of-way until such time as the approval letter has been issued to the Contractor.

Attachments

c: BCR&P Manual Holders
Consultant Firms
CABomar, Chief, Railroad Agreements Section
BFelker, Construction Program Manager

GUIDELINES FOR PREPARATION OF A BRIDGE DEMOLITION AND REMOVAL PLAN FOR STRUCTURES OVER RAILROAD



UNION PACIFIC RAILROAD

OFFICE OF CHIEF ENGINEER DESIGN
1416 DODGE ST.
OMAHA. NE 68179

BRIDGE CONSTRUCTION MEMO 124-3
ATTACHMENT No. 1 (5/99)
Sheet 1 of 13

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I. GENERAL

A. The Contractor's work shall in no way impede the train operations of the Union Pacific Railroad.

B. The Contractor shall develop a work plan assuming that minimal track windows will be available.

C. The Contractor shall be responsible for planning and executing all procedures necessary to remove the overhead bridge in a safe and controlled manner.

D. The Railroad's tracks and property shall be protected at all times.

E. The contractor shall ensure the area immediately adjacent to operational tracks shall remain free from stumble or like hazards to the ground Railroad personnel to prevent injuries. Open excavations shall be in accordance with current CE Drawing 106613 and shall be protected by appropriate fencing.

F. The words "demolition" and "removal" will be used interchangeably.

G. All removed materials shall be disposed of outside the Railroad right-of-way at no expense to the Railroad.

H. No work is allowed within 50 feet of the nearest rail when trains pass the work site.

I. Staged demolition of the portions of structure immediately adjacent to operational tracks will not jeopardize the integrity of the structure over said tracks until actual removal of the portion of the structure over the tracks is being done.

J. A flagman is required when any work is performed within 25 feet of the nearest rail.

K. No blasting will be permitted on Railroad's right-of-way.

II. BRIDGE REMOVAL PLAN

A. The Contractor shall submit a complete Bridge Removal Plan to the Railroad. The Bridge Removal Plan shall include details, procedures and the sequence of staged removal of the bridge, including all steps necessary to remove the bridge in a safe and controlled manner.

BRIDGE CONSTRUCTION MEMO 124-3
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B. The Contractor shall submit to the Railroad; three (3) complete sets of the Bridge Removal Plan for review and comments. The Plan shall be sealed by a Civil or Structural Engineer registered in the state where the proposed demolition will take place. A minimum of three (3) weeks shall be allowed for the Railroad's review after the complete submittal is received. No removal operations will be permitted over the Railroad right of way until the submitted material has been reviewed and comments provided.

C. Review and comment of the Removal Plan by the Railroad will not relieve the Contractor of the ultimate responsibility and liability for the demolition of the structure.

D. The Removal Plan shall include the following:

1) Plan, elevation and location of the bridge, and the locations of any access roads needed for movement of the equipment. The as-built drawings may be used for the submittal provided the removal steps are clearly marked and legible.

2) Indicate the position of all railroad tracks below the bridge and identify each track as mainline, siding, spur, etc.

3) Bridge removal sequence and procedures for entire bridge including the staging for the removal of the superstructure and substructure.

4) List type and number of equipment required and their locations during demolition operations.

5) Locations and types of temporary supports, shoring or bracing required. These members shall be designed to meet Union Pacific Railroad current standard drawing 106613 "General Shoring Requirements", "Guidelines for Design and Construction of Falsework for Structures Over Union Pacific Railroad", "Guidelines for Design and Construction of Shoring Adjacent to Active Railroad Tracks", and the appropriate local and national building and design code requirements.

6) The proposed vertical and horizontal clearance from all tracks to the temporary and permanent supports. The minimum vertical and horizontal clearances shall be as per attached frame protection details.

7) If any temporary supports interfere with the natural drainage along the Railroad right-of-way, a temporary drainage plan shall be submitted for review and comment prior to constructing temporary supports. The proposed drainage plan shall route all drainage away from the railroad tracks.

BRIDGE CONSTRUCTION MEMO 124-3
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8) Details, limits, and locations of protective covers or other measures proposed to be used to protect the tracks. This includes any shields or other measures that will protect the tracks from falling debris during removal of the overhead bridge and from any debris rolling down the side slopes or otherwise coming into the area around the tracks which could affect train operations. Design loads, including impact loads, shall be noted. In addition equipment should be on site capable of removing debris and track shield from operational tracks.

9) All procedures necessary to remove the bridge in a safe and controlled manner. The estimated time for complete removal over the tracks shall be noted.

10) All overhead and underground utilities in the area affected by removal of the bridge shall be located on the drawings, including any fiber optic, railroad signal, and communication lines.

11) The location and details of track crossings required for moving of the equipment across the railroad tracks.

12) Limits of demolition of substructures.

13) Details of on-site fire suppression.

III. PROCEDURE

A. During removal operations the remaining structure shall be stable during all stages of the removal operations.

B. Prior to proceeding with bridge removal the sealing Civil or Structural Engineer, or his authorized representative working for the Contractor, shall inspect the temporary support shoring, including temporary bracing and protective coverings, for conformity with the working drawings. The Engineer shall certify in writing to the Railroad that the work is in conformance with the drawings and that the materials and workmanship are satisfactory. A copy of this certification shall be available at the site of work at all times.

C. Coordinate the removal schedule with the Railroad. All the removal work within the track area shall be performed during the time windows when the trains are not passing the work site.

D. All substructures shall be removed to at least 3 feet below the final finished grade or at least 2 feet below base of rail whichever is lower, unless otherwise specified by the Railroad.

E. All debris and refuse resulting from the work shall be removed from the right of way by the contractor and the premises left in a neat and presentable condition.

F. The work progress shall be reviewed and logged by the Contractor's Engineer. Should an unplanned event occur, the Contractor shall inform the Railroad and submit procedure to correct or remedy the occurrence.

G. Preferably all demolition and beam removal shall be from above. In the case that the beams require removal from below, the beams may temporarily straddle the tracks. The following steps shall be taken:

1) The work shall be scheduled with the Railroad's Service Unit Superintendent subject to the Railroad's operational requirements for continuos train operations. The beams removed in sufficient time for train passage.

2) The tracks shall be protected and no equipment placed on the tracks.

3) The beams shall be blocked and not come in contact with the tracks. Blocking shall not be placed on the tracks.

4) The beams and all equipment will be moved a minimum of 15 feet from the nearest rail of the tracks when a train is passing.

IV. TRACK PROTECTION

A. The track protective cover shall be constructed before beginning bridge removal work and may be supported by falsework or members of the existing structure. See the attached Track Shield Detail and Frame Protection Detail for additional requirements. Types of protective covers that may be acceptable methods for protecting the tracks are:

1) A decking supported by the bridge or a suspended cover from the bridge above the track clearance envelope.

2) A track shield cover over the tracks per the attached detail.

3) A framed cover outside the track clearance envelope.

4) A catcher box or loader bucket under decking and parapets overhanging the exterior girders.

B. Construction equipment shall not be placed on the tracks unless tracks are protected.

C. Temporary haul road crossings shall are of either Section Timbers or Precast Concrete Panels. The type of crossing shall be determined by the Manager of Industry and Public Projects. Solid timbers or ballast with timber headers shall be used between multiple tracks. If temporary crossing is accessible to public crossing shall be protected with barricades or locked gates when contractor is not actively working at the site or weekends.

D. Track protection is required for all equipment including rubber tired equipment operating within 25 ft. or over the tracks.

V. CRANES

A. When cranes are operated near the tracks the following is required:

1) Only cranes with the capacity to handle the loads may be used. Front end loaders and backhoes cannot be used to lift over the tracks.

2) The Contractor shall verify that the foundations under the crane can support the loads.

3) The size and material type of crane mats shall be submitted to the Railroad for review and comment. No mat substitution will be allowed. The mats shall be rigid and of sufficient capacity to distribute the crane loads and prevent tipping of the crane.

4) Installation of temporary track crossings for equipment shall be scheduled with the Manager of Industry and Public Projects .

5) Additional track protection is required when crossing with a crane. The protection methods shall be submitted to the Railroad for review and comment.

6) Equipment shall not place outriggers on the tracks or ballast.

7) Cranes shall not be placed within the track clearance envelope without flagman protection.

VI. CUTTING TORCHES

A. When a cutting torch is used near the tracks or any timber, the following steps shall be taken:

1) Fire suppression equipment is required on-site.

2) Do not use a torch over, between, or adjacent to the tracks unless a steel plate protective cover is used. Care shall be taken to make certain the use of a steel plate does not come in contact with the rails. See "Track Shield Details" for other requirements. Details of the shield shall be submitted to the Railroad for approval..

3) Wet the ties and other timber below the cutting area.

4) Monitor the work site for at least three hours after cutting for a smoldering fire.

B. Extensive overhead cutting will not be performed over the track area without the proper fire suppression equipment on-site and proper protection.

VII. UTILITIES

A. The demolition operations shall be planned such that the utility lines are operating safely at all times. The utility lines shall be protected if affected by demolition operations. All the work associated with utility lines should be coordinated by the contractor with the respective utility companies.

VIII. HAZARDOUS MATERIAL

A. If any hazardous materials are found, provide material protection as specified in local hazardous material codes and immediately contact the Railroad.

BRIDGE CONSTRUCTION MEMO 124-3
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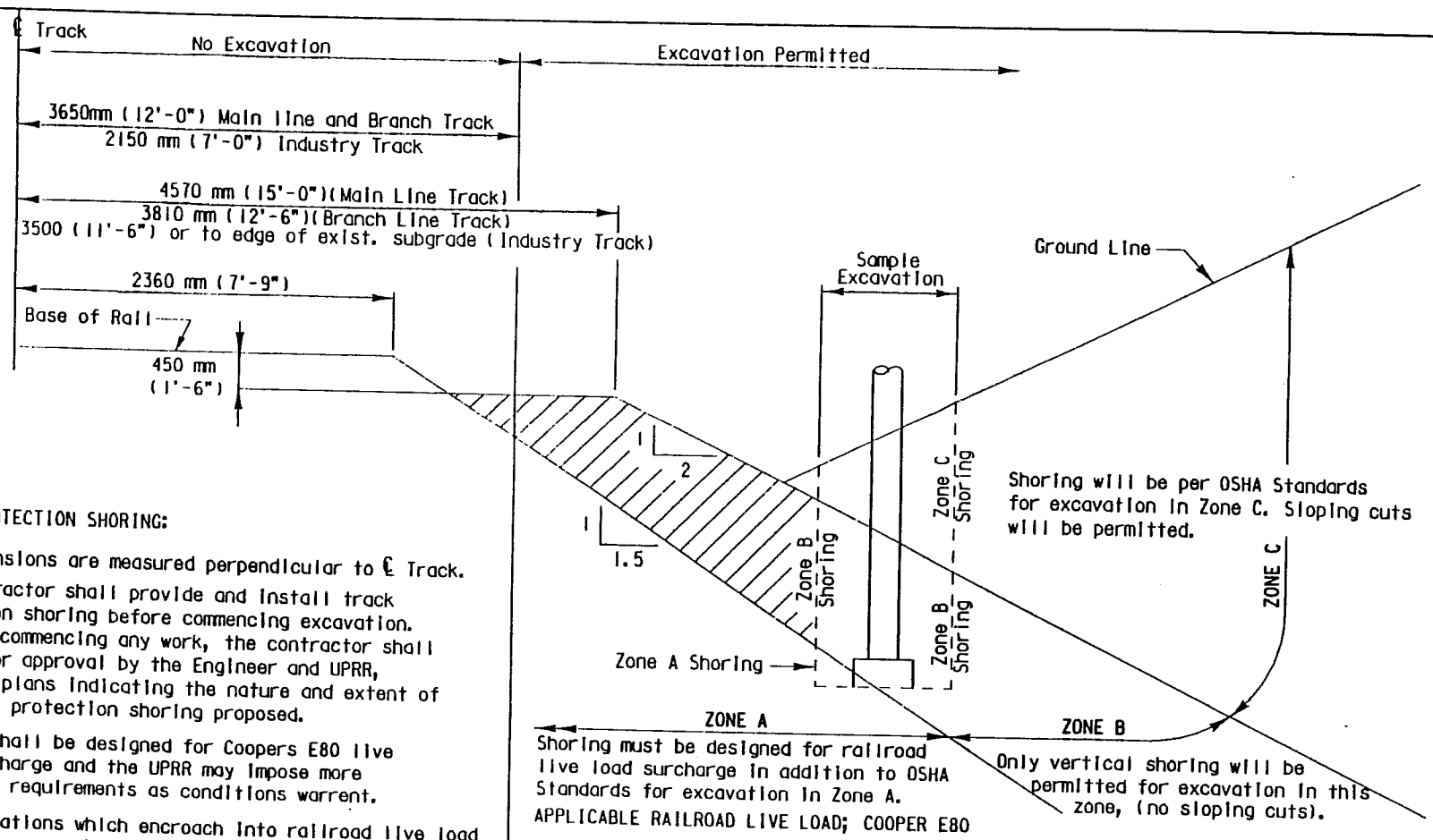
APPENDIX

- U.P.R.R. STANDARD DRAWING 106613
- TRACK SHIELD DETAIL
- FRAME PROTECTION DETAILS



TRACK PROTECTION SHORING:

All dimensions are measured perpendicular to ℄ Track.

The contractor shall provide and install track protection shoring before commencing excavation. Prior to commencing any work, the contractor shall submit for approval by the Engineer and UPRR, detailed plans indicating the nature and extent of the track protection shoring proposed.

Shoring shall be designed for Coopers E80 live load surcharge and the UPRR may impose more stringent requirements as conditions warrent.

For excavations which encroach into railroad live load surcharge zone, shoring plans will be accompanied by a copy of the design calculations, and both must be stamped by a registered professional engineer.

Design of shoring shall comply with UPRR guidlines for design and construction of shoring adjacent to active railroad tracks.

TRACK PROTECTION SHORING REQUIREMENTS

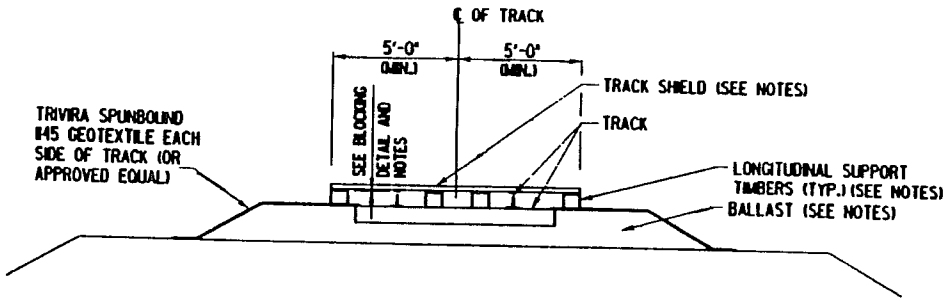
UNION PACIFIC RAILROAD

GENERAL SHORING REQUIREMENTS

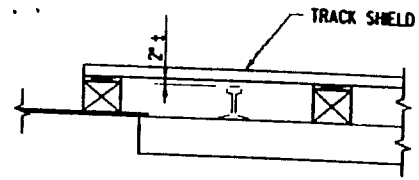
OFFICE OF CHIEF ENGINEER DESIGN

DATE: 3-31-98 REDRAWN

C.E. 106613



TRACK SHIELD DETAIL
FOR DEBRIS FALLING FROM BRIDGE DECK REMOVAL
(WHEN TRACK TIME WINDOW IS AVAILABLE)



BLOCKING DETAIL

NOTES:

1. A FLAGMAN IS REQUIRED AT ALL TIMES DURING THE USE OF A TRACK SHIELD.
2. THE TRACK SHIELD SHALL BE DESIGNED BY THE CONTRACTOR AND SHALL BE OF SUFFICIENT STRENGTH TO SUPPORT THE ANTICIPATED LOADS, INCLUDING IMPACT. THE SHIELD SHALL PREVENT ANY MATERIALS, EQUIPMENT OR DEBRIS FROM FALLING ONTO THE RAILROAD TRACK. ADDITIONAL LAYERS OF MATERIALS SHALL BE FURNISHED AS NECESSARY TO PREVENT FINE MATERIALS OR DEBRIS FROM SIFTING DOWN UPON THE TRACK.
3. THE SHIELD SHOULD PREFERABLY BE PREFABRICATED AND FURNISHED WITH LIFTING HOOKS TO SIMPLIFY REMOVAL.
4. THE SHIELD SHALL BE OF SUFFICIENT STRENGTH TO SPAN BETWEEN IT'S SUPPORTS WITHOUT BEARING UPON THE RAILS AND TO WITHSTAND DROPPING RUBBLE.
5. BEFORE REMOVAL, THE SHIELD SHALL BE CLEANED OF ALL DEBRIS AND FINE MATERIAL.
6. THE TRACK SHIELD SHALL EXTEND AT LEAST 20 FEET BEYOND THE LIMITS OF DEMOLITION TRANSVERSE TO THE EDGE OF THE BRIDGE.
7. LONGITUDINAL SUPPORT TIMBERS FOR THE SHIELD SHALL NOT EXTEND ABOVE THE TOP OF RAIL WHEN THE SHIELD IS REMOVED. BLOCKING FROM THE TOP OF RAIL TO THE BOTTOM OF THE SHIELD MAY BE ATTACHED TO THE SHIELD. REMAINING TIMBERS SHALL BE ANCHORED.
8. FOR TRAIN PASSAGE, THE RUBBLE SHALL BE REMOVED TO A MINIMUM OF 8' 6" FROM THE NEAREST RAIL AND TO AN ELEVATION NO HIGHER THAN THE TOP OF RAIL.
9. AT THE END OF THE DAY, THE RUBBLE SHALL BE REMOVED COMPLETELY TO A MINIMUM OF 10' 0" FROM THE NEAREST RAIL AND DOWN TO ORIGINAL GRADE.
10. CARE SHALL BE TAKEN TO NOT PLACE METAL ACROSS THE TRACK RAILS. RAILROAD COMMUNICATIONS ARE SENT THROUGH THE RAILS AND WILL BE DISRUPTED BY A SHORT BETWEEN RAILS.
11. DETAILS SHOWN APPLY FOR TIMBER TIES. SPECIAL DETAILS ARE REQUIRED FOR CONCRETE TIES.

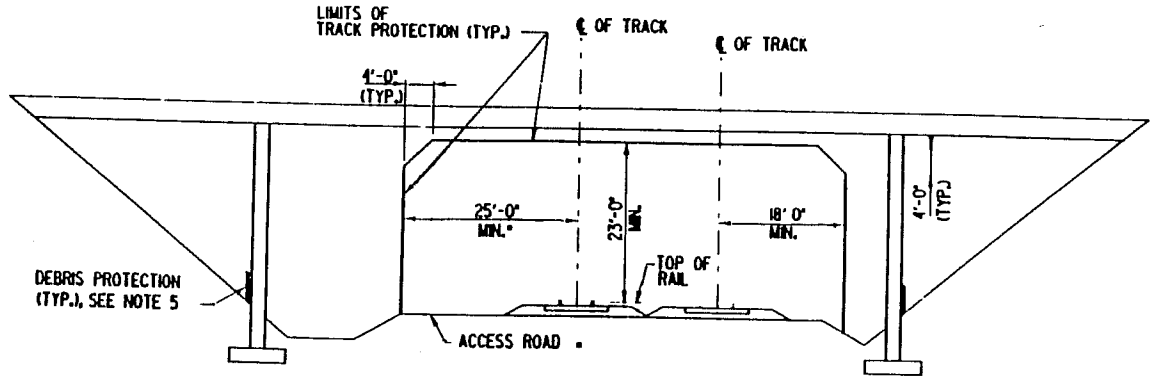
UNION PACIFIC RAILROAD

TRACK SHIELD DETAIL

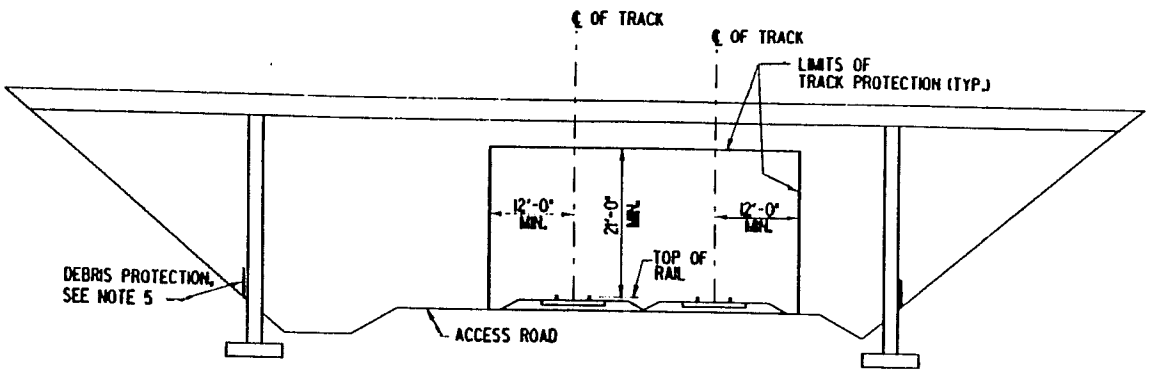
OFFICE OF CHIEF ENGINEER DESIGN

DATE: 3-31-98

SHEET 1 OF 1



BRIDGE ELEVATION
STANDARD LIMITS OF PROTECTION FOR FRAME PROTECTION



BRIDGE ELEVATION
MINIMUM LIMITS OF PROTECTION FOR FRAME PROTECTION
(SPECIAL PERMISSION REQUIRED, SEE NOTE 1)

* IF NO ACCESS ROAD, USE MIN. DIMENSION FROM OTHER SIDE OF DETAIL.

NOTES:

1. THE STANDARD LIMITS OF PROTECTION NOTED ARE THE MIN. CLEARANCES ALLOWED WITHOUT SPECIAL PERMISSION FROM THE RAILROAD. THE REDUCED CLEARANCES NOTED MAY BE ALLOWED BY THE RAILROAD. SPECIAL PERMISSION FOR THE REDUCED CLEARANCES IS REQUIRED FROM THE RAILROAD SERVICE UNIT SUPERINTENDENT.
2. THE PROTECTION FRAME SHALL AS A MINIMUM MATCH THE DEMOLITION LIMITS SHOWN AND EXTEND PAST THE BRIDGE WIDTH AS SHOWN ON THE ATTACHED DEMOLITION PLAN SHEET.
3. FOR ADDITIONAL CLEARANCE AND PROTECTION INFORMATION, SEE UNION PACIFIC RAILROAD STANDARD DRAWING NO. 0035
4. THE PROTECTION FRAME SHALL PREVENT DEMOLITION DEBRIS, DUST AND FINE MATERIAL FROM FALLING ONTO THE RAILROAD TRACKS, ACCESS ROAD OR TRAINS. THE FRAME SHALL BE DESIGNED BY THE CONTRACTOR TO SUPPORT THE ANTICIPATED DEMOLITION LOADS, AND IN ACCORDANCE WITH UNION PACIFIC GUIDELINES FOR DESIGN OF FALSEWORK FOR STRUCTURES OVER THE RAILROAD.
5. DEBRIS PROTECTION IS REQUIRED NEAR THE BASE OF THE SIDE SLOPES AND ADJACENT TO ROADS USED BY DEMOLITION EQUIPMENT TO PREVENT DEBRIS FROM ROLLING ONTO THE TRACK, ACCESS ROAD OR DITCH. USE TIMBERS AS REQUIRED TO STOP LARGE PIECES OF ROLLING DEBRIS.
6. ANY ACTIVITY WITHIN 25 FEET OF THE NEAREST RAIL OF A TRACK REQUIRES A FLAGMAN.

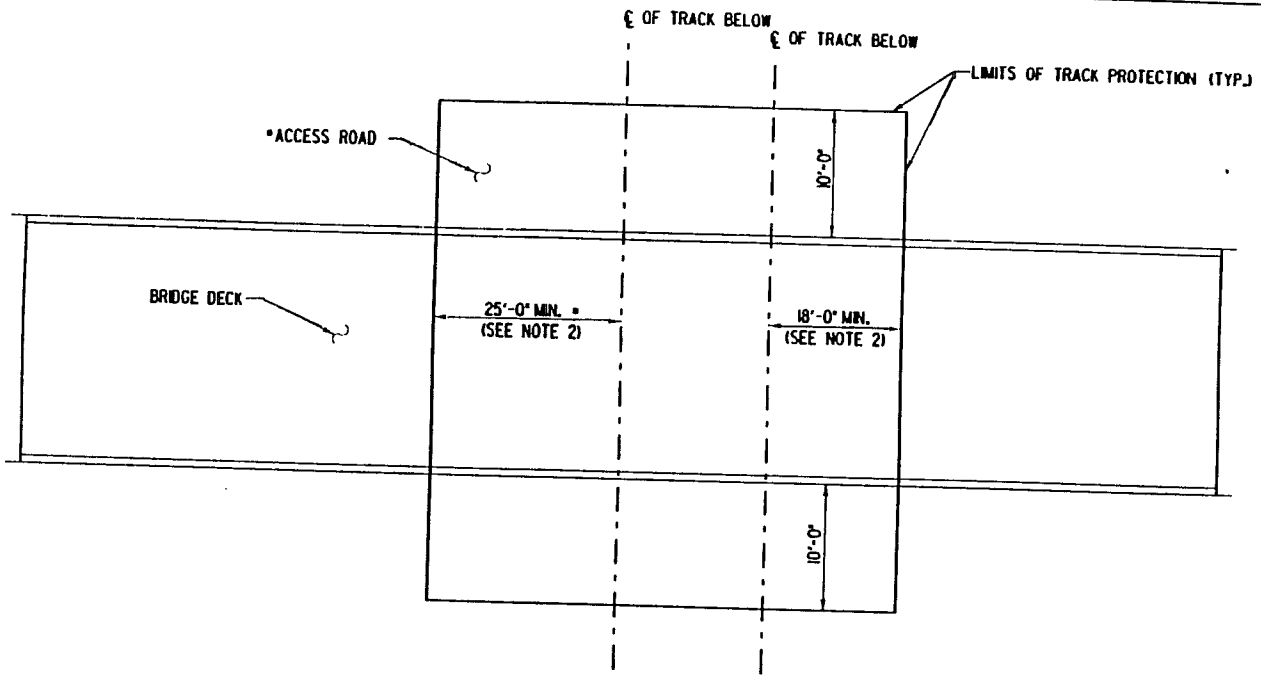
UNION PACIFIC RAILROAD

FRAME PROTECTION DETAILS

OFFICE OF CHIEF ENGINEER DESIGN

DATE: 3-31-98

SHEET 1 OF 2

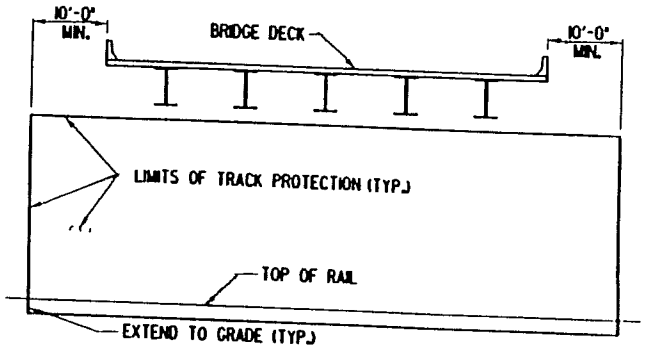


BRIDGE PLAN
STANDARD LIMITS OF PROTECTION FOR FRAME PROTECTION

* IF NO ACCESS ROAD, USE MIN. DIMENSION FROM OTHER SIDE

NOTES:

1. SEE GENERAL NOTES ON BRIDGE ELEVATION SHEET
2. STANDARD LIMITS OF PROTECTION ARE SHOWN. FOR MIN. LIMITS OF PROTECTION DIMENSIONS, SEE BRIDGE ELEVATION, MINIMUM LIMITS OF PROTECTION.



BRIDGE DECK CROSS SECTION
STANDARD LIMITS OF PROTECTION

* IF NO ACCESS ROAD, USE MIN. DIMENSION FROM OTHER SIDE

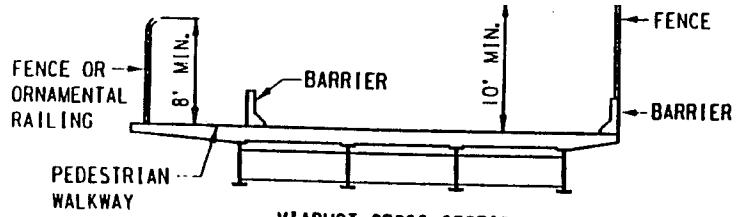
UNION PACIFIC RAILROAD

FRAME PROTECTION DETAILS

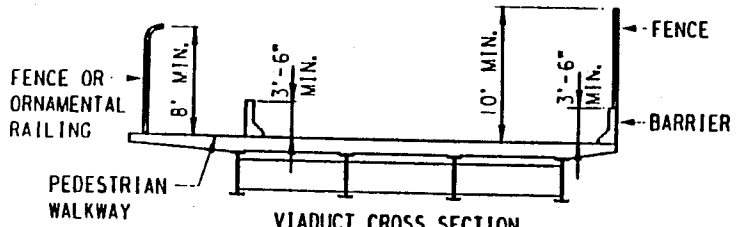
OFFICE OF CHIEF ENGINEER DESIGN

DATE: 3-31-98

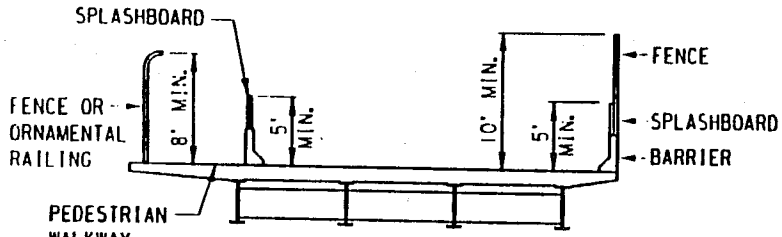
SHEET 2 OF 2



VIADUCT CROSS SECTION
NO SNOW REMOVAL AREAS



VIADUCT CROSS SECTION
WITH 3'-6" SOLID BARRIER AND FENCE
FOR SNOW REMOVAL AREAS



VIADUCT CROSS SECTION
WITH SPLASH BOARD AND FENCE
FOR SNOW REMOVAL AREAS

GENERAL

Fence shall be provided as indicated on the cross sections and elevation view on both sides of the viaduct in ALL new or modified structures.

Splashboards or solid 3'-6" high barrier rail shall be provided as indicated on the cross sections and elevation view on both sides of the viaduct in ALL new or modified structures where snow removal is being performed.

Lights are to be installed on the underside of the viaduct where shadows cast by the structure would interfere with Railroad operations.

Slope paving shall be provided where end slopes exceed 2 horizontal to 1 vertical.

Falsework for costruction of overhead structures shall comply to UPRR guidelines.

Demolition of existing overhead structures shall comply to UPRR guidlines.

Temporary shoring shall be designed in accordance with UPRR's Shoring Requirements (Drawing No. 106613) and UPRR guidlines.

Applicant shall be responsible for identification, location, and protection of existing utilities.

Contact UPRR's "Call Before You Dig" at least 48 hours prior to commencing work at 1-800-336-9193 to determine location of fiber optics.

Exceptions to these standards must be approved by UPRR's Chief Engineer Design.

CLEARANCES

Minimum vertical clearance shall be 23 feet above the plane of top-of-rails. Additional clearance may be required for construction purposes or if sag of vertical curve must be adjusted or if future track raise for flood considerations or maintenance is probable.

Minimum horizontal clearances, measured at right angle from centerline of track, shall be as shown in elevation view.

Minimum construction clearances shall be 21 feet vertical above the plane of top-of-rails and 12 feet horizontal at right angle from centerline of track.

FUTURE TRACKS

Space is to be provided for one or more future tracks as required for long range planning or other operating requirements. Where provision is made for more than two tracks, space is to be provided for access road on both sides of tracks.

PIERS

Pier protection (crash walls) shall be provided in accordance with AREA Chapter 8, Part 2.1.5 for piers within 25 feet of the centerline of track.

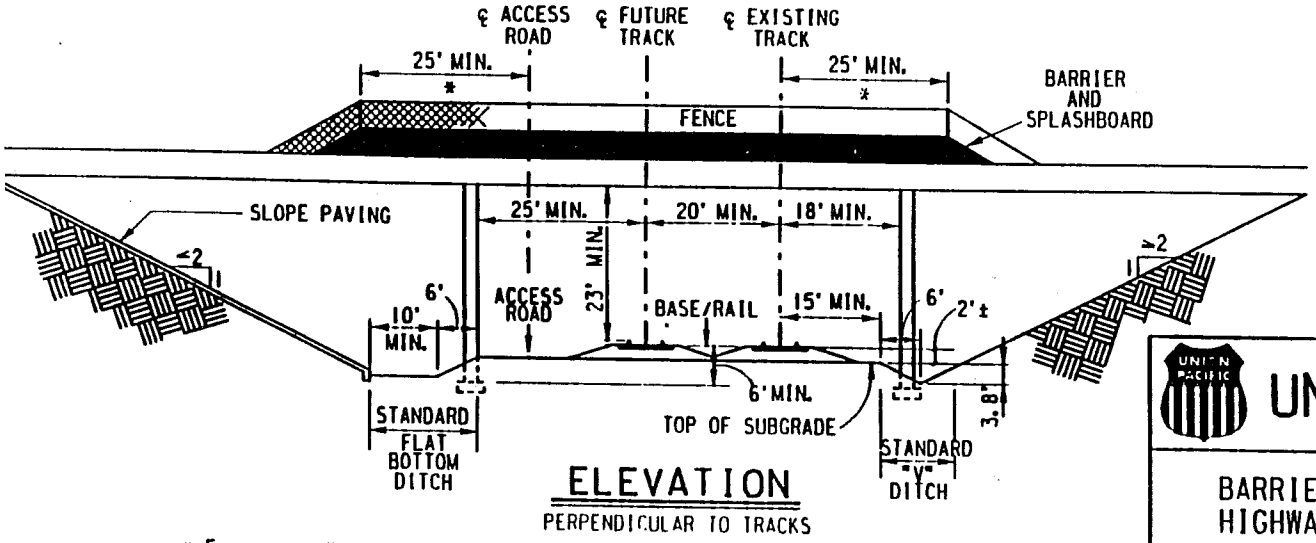
Top of footings within 25' from centerline of track shall be a minimum of 6 feet below base of rail and a minimum of 1 foot below flow line of ditch.

DRAINAGE

Drainage from the overpass shall be diverted away from UPRR's tracks and not discharged onto the tracks or roadbed.

A standard "V"-shaped or flat-bottom ditch shall be provided on each side of the tracks as necessary.

Culverts may be installed on opposite side of column from track in lieu of standard Railroad ditches when approved by Chief Engineer Design. Maintenance of culverts is to be at applicant's expense.



ELEVATION
PERPENDICULAR TO TRACKS

* Fences, splashboards, or solid barriers if required shall extend 25ft. beyond centerline of outer most track or access roadway.

UNION PACIFIC RAILROAD

BARRIERS AND CLEARANCES TO BE PROVIDED AT HIGHWAY, STREET, AND PEDESTRIAN OVERPASSES

OFFICE OF CHIEF ENGINEER DESIGN

REVISED: MAR. 31. 1998

STD DWG 0035

BRIDGE CONSTRUCTION MEMO 124-3
ATTACHMENT No. 2 (5/99)
Sheet 1 of 1

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MEMO NO.	ISSUE DATE	TITLE
130-1.0	12/09/2011	Foundation Testing Branch (FTB)
130-2.0	06/30/2014	Pile Load Tests
130-3.0		(Blank – Removed 06-30-14)
130-4.0	06/30/2014	Pile Driving Acceptance Criteria
130-5.0	12/09/2011	Steel H-Pile Lugs
130-5.0 Att. No. 1	12/09/2011	Steel H-Pile Lug Drawing
130-6.0	06/30/2014	Payment for Piling
130-7.0	06/30/2014	CIDH Concrete Piling
130-8.0	12/09/2011	CIDH Pile Mitigation Committee
130-9.0	06/30/2014	CIDH Pile Installation Plan and Concrete Test Batch
130-10.0	06/30/2014	Testing of CIDH Piling
130-11.0	06/30/2014	Simple Repair of CIDH Piling
130-12.0	06/30/2014	Mitigation of CIDH Piling
130-13.0	06/30/2014	CIDH Pile Information Submittal
130-14.0	06/30/2014	Slurry Test Kits for CIDH Piling
130-15.0	12/09/2011	Approved Synthetic Drilling Slurries
130-16.0	06/30/2014	Settlement Platforms
130-17.0		(Blank)
130-18.0		(Blank – Removed 06-30-14)
130-19.0	06/30/2014	Reference to the Foundation Manual – Appendix K for Construction Checklists for Foundation Work
130-20.0	02/09/2011	CIDH Pile Preconstruction Meeting
130-20.0 Att. No. 1	12/09/2011	CIDH Pile Preconstruction Meeting Agenda/Minutes
130-20.0 Att. No. 1.1	12/09/2011	Attendance Sheet CIDH Pile Preconstruction Meeting
130-20.0 Att. No. 1.2	12/09/2011	CIDH Wet vs. Dry (Inspection Tube Requirements)
130-20.0 Att. No. 1.3	12/09/2011	CIDH Drilling & Concrete Placement
130-20.0 Att. No. 1.4	12/09/2011	Synthetic Slurry Test Record

130-20.0 Att. No. 1.5	12/09/2011	Concreting Yield Plot
130-20.0 Att. No. 1.6	12/09/2011	GGL Inspection Tube Verification
130-20.0 Att. No. 1.7	06/30/2014	CIDH Pile Acceptance Test Request Form
130-20.0 Att. No. 1.8	12/21/2012	Pile Design Data Form
130-20.0 Att. No. 1.9	12/09/2011	Pile Mitigation Procedure
130-20.0 Att. No. 1.10	12/09/2011	Mitigation Timeline – General Draft
130-21.0	06/30/2014	CIDH Pile Non-Standard Mitigation Meeting
130-22.0	06/30/2014	Seal Courses

Robert A. Stott, Deputy Division Chief
Structure Construction

Denotes the document is a Bridge Construction Bulletin

Volume II

FOUNDATION TESTING BRANCH

The Foundation Testing Branch, Office of Geotechnical Support, Geotechnical Services, provides statewide foundation testing services which include:

- Static load tests for compression, tension, and lateral capacities of piles.
- Pile Dynamic Analysis.
- Nondestructive testing of Cast-In-Drilled-Hole (CIDH) piles (Gamma-Gamma and Crosshole Sonic Logging).
- CIDH Pile Mitigation Review.
- Pile Drivability Studies.
- Field acceptance criteria by wave equation analysis.
- Sonic Caliper Testing.
- Shaft bottom inspection utilizing the Shaft Inspection Device (SID).
- Geotechnical Testing Consultation.

For contracts that require any of the listed testing services, contact the Foundation Testing Branch (FTB) in Sacramento. Foundation testing request forms can be downloaded from the FTB website located at http://www.dot.ca.gov/hq/esc/geotech/request.htm .

Pile Load Tests

The Standard Specifications [1](SS) explain that load testing of test piles must be complete before production piles for that structure or the specified control locations are drilled, cast, cut to length, or driven.

If control locations apply, this information is in the Special Provisions. Test pile layout, special test pile design details, and anchorage are shown on the plans. The test requirements and details are developed for each contract, so early review of the plans and specifications are very important, as is communication and coordination with the Foundation Testing Branch (FTB).

The Standard Specifications requires that the Engineer be notified at least ten days before drilling or driving piles are load tested. The FTB requests that they be notified at least three weeks prior to the start of any drilling or pile driving. See BCM 130-1.0, *Foundation Testing Branch*.

- For testing driven piling, most contracts will include dynamic monitoring during driving. The FTB will need to instrument the test pile before it is lifted for driving.

- Testing of Cast-in-Drilled-Hole (CIDH) piling may require placing a load cell (with instrumentation) at the bottom of the drilled hole prior to placing concrete. The FTB will need to instrument a test pile while the rebar cage is being tied

Assistance from the Contractor in the installation, operation, and removal of load beams, jacks, bearing plates, and other equipment will be necessary. Per the Standard Specifications[1], this work will be paid for as extra work.

Note that the test and anchor piles may have different spacing and tip elevations than the production piles. The Engineer's Estimate includes the extra lengths for test piles.

Full compensation for re-driving dynamically monitored piling, dewatering during monitoring, and for removing piling not incorporated into the permanent work is included in the price paid for pile driving.

Additional information on pile load tests can be found in Chapter 8 of the *Foundation Manual* and in Section 4.3 of the *Foundation Testing, Management, Practices, and Procedure Manual*.

[1] 2010 SS, Section 49-1.01D(3), *Load Test Piles*, or 2006 SS, Section 49-1.04, *Load Test Piles*.

Pile Driving Acceptance Criteria

Projects with standard driven pile foundations specify the *Gates Formula* to determine nominal driving resistance. The *Gates Formula* is:

$$R_u = (7 * (E_r)^{1/2} * \log_{10} (0.83 * N)) - 550$$

Where:

R_u = Calculated nominal driving resistance in kiloNewtons.
E_r = Energy rating of hammer at observed field drop height in Joules.
N = Number of blows per 300 millimeters (maximum of 96).

In US Customary units:

$$R_u = (1.83 * (E_r)^{1/2} * \log_{10} (0.83 * N)) - 124$$

Where:

R_u = Calculated nominal driving resistance in Kips.
E_r = Energy rating of hammer at observed field drop height in foot-pounds.
N = Number of blows per foot (maximum of 96).

In order to verify that the proposed hammer can develop the required minimum energy as required by the specifications[1], use the manufacturer's maximum energy rating and the nominal driving resistance to calculate the maximum acceptable blow count (not exceeding 96 blows per foot which is equivalent to a penetration rate of not less than 1/8 inch per blow). Hammer data is typically submitted by the Contractor and can be found at the hammer manufacturer website or by contacting the Foundation Testing Branch (FTB). See BCM 130-1.0, *Foundation Testing Branch*.

When calculating the number of blows for the required nominal driving resistance, E_r can be calculated by multiplying the hammer ram weight by the observed stroke.

A simple spreadsheet (PileEquation-Gates.xls) used to calculate the N value can be found on the SC Homepage under *Downloads/Forms*[2]:

Appendix E of the *Foundation Manual* provides examples for calculations for minimum hammer energy, establishing a blow count chart, battered pile blow count chart, and other examples.

[1] 2010 SS, Section 49-2.01C(2), *Driving Equipment*, or 2006 SS Section 49-1.05, *Driving Equipment*.
[2] http://onramp.dot.ca.gov/hq/oscnet/downloads/forms.htm

It is important to note that:

- The *Gates Formula* uses nominal values. Nominal resistance and nominal driving resistance of a given pile are shown in the Pile Data Table on the contract plans.
- The nominal driving resistance is always equal to or greater than the nominal resistance. This is because the nominal driving resistance accounts for driving resistance through unsuitable penetrated soil layers (very soft, liquefiable, scourable, etc.) which do not contribute to the design nominal resistance.
- Even under ideal hammer operations, the energy dissipation from impact and losses to the hammer mechanism may greatly reduce the actual energy delivered to the pile during driving. Additional losses may occur due to improper or inadequate hammer use, changing fuel setting, using interchangeable ram, etc. Be aware of reductions in actual hammer energy. Using a false high hammer energy value in the *Gates Formula* will give false high nominal resistance results.

When requested by the geotechnical designer, the special provisions will include a driving system submittal section. When submitted by the Contractor, forward this submittal to the FTB for review.

The *Gates Formula* should not be used when the driven pile diameter is greater than or equal to 18-inches or the required nominal driving resistance exceeds 600 kips. In such cases the Special Provisions will include appropriate test method(s) for the verification of the axial nominal resistance during installation and the pile acceptance criteria. These tests include pile dynamic monitoring and/or a pile load test. Contact the FTB for dynamic monitoring or a pile load test.

Volume II

STEEL H-PILE LUGS

Pile lugs are welded onto steel H-piles prior to driving. The lug increases the friction resistance of the pile so that the bearing is achieved with a shorter pile. Welding of steel pilings, including attachment of lugs, must conform to Standard Specifications[1] (SS). The detail for a pile lug is shown in Attachment No. 1.

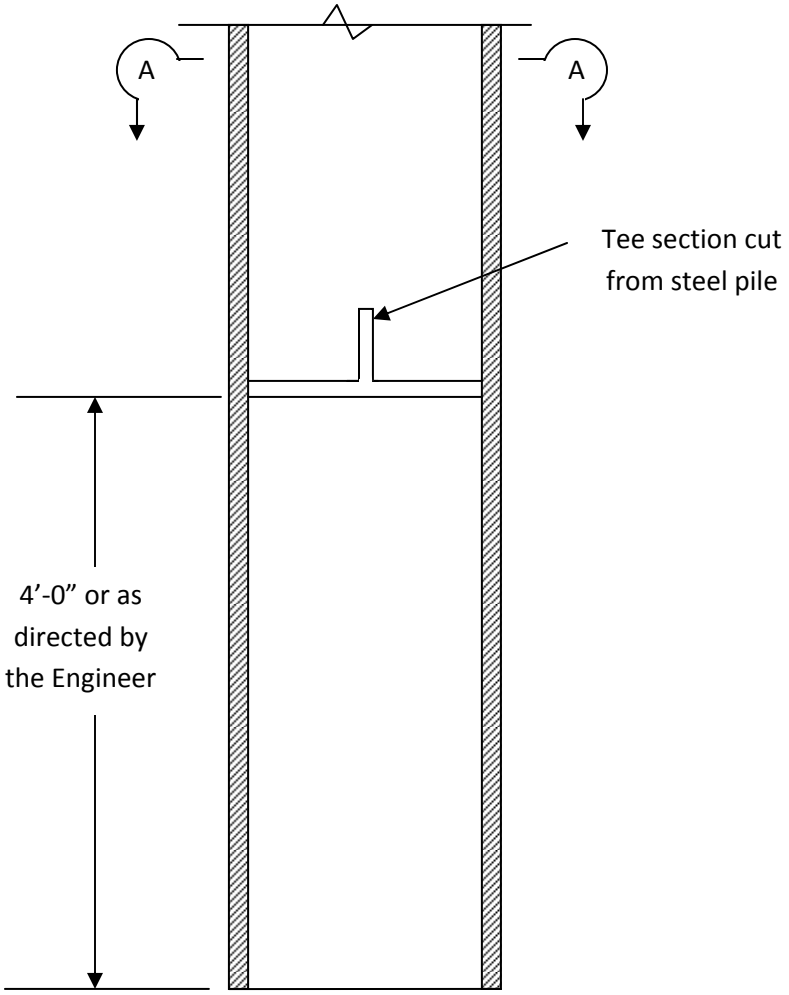
When recommended by Geotechnical Services, the pile lug detail will be shown on the plans. Review the special provisions for measurement and payment. Lugs that are shown on the plans are paid as furnishing piling per the SS.

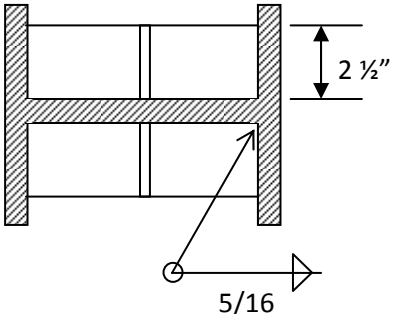
When lugs are not shown on the plans, and when the piles are driving longer than anticipated, immediately contact Geotechnical Services. An option to the use of lugs is to let the piles set up and to re-drive (and check bearing) a day or two later.

The Contractor may request lugs or the Engineer may order lugs. Savings from reducing the length of the pile must be compared with the cost for furnishing, welding and performing welding quality control on the lugs. Lugs might not be economical when there is no other welding on the job.

Lugs installed at the direction of the Engineer are paid for as change order work as stated in the SS[2].

[1]SS 2010, Section 11 or SP 2006, Section 8-3.
[2]SS 2010, Section 49-2.01C(1) or SS 2006, Section 49-6.02.





SECTION A-A

PILE LUG

Measurement and Payment for Piling

Measurement and payment clauses are in the *Standard Specifications (SS)*, the *Special Provisions (SP)*and the *Construction Manual*. Review these documents.

Measurement

The SS[1] specify how piles are measured and paid for. However, the requirements of the SS vary depending on the version used when the contract was written.

Contracts using Section 49-6.01 of the 2006 SS provide for measurement of piling as follows:

> *The length of timber, steel, and precast prestressed concrete piles, and of cast-in-place concrete piles consisting of driven shells filled with concrete, shall be the greater of the following:*
>
> *A. The total length in place in the completed work, measured along the longest side, from the tip of the pile to the plane of pile cut-off.*
> *B. The length measured along the longest side, from the tip elevation shown on the plans or the tip elevation ordered by the engineer, to the plane of pile cut-off.*

Piling that extends beyond the tip elevation shown on the plans, as ordered by the Engineer, to meet design requirements, will be measured under the provisions of Part A; while piling that fails to reach the tip elevation shown on the plans, but has been determined to be suitable for the design, will be measured in accordance with Part B.

Contracts using amended versions of Section 49-6.01 of the 2006 SS provide for measurement of piling as follows:

> *The length of timber, steel, and precast prestressed concrete piles, and of cast-in-place concrete piles consisting of drive shells filled with concrete, shall be measured along the longest side, from the tip elevation shown on the plans to the plane of pile cut-off.*

Contracts using revised versions of Section 49-2.01D of the 2010 SS provide for measurement of piling as follows:

> *Furnish piling is measured along the longest side of the pile from the specified tip elevation shown on the plans to the plane of pile cutoff.*

[1] 2010 SS, Section 49-2.01D, *Payment*, or 2006 SS, Section 49-6, *Measurement and Payment*.

Piling that fails to reach tip elevations shown on the plans, but has been determined to be adequate and approved by the Designer, will be measured along the longest side, from the tip elevation shown on the plans to the plane of cut-off elevation.

Payment

Materials on Hand

Bridge Construction Memo (BCM) 6-4.0, *Partial Payments*, addresses the differences between *Materials on Hand but not yet incorporated in the work*, and payments for *furnishing materials*. Refer to BCM 6-4.0 prior to making payments for piling.

When the SP qualify the material for *Materials on Hand* and it does not meet the requirements for "furnishing", payment may be made as *Materials on Hand* at the Contractor's request.

Precast concrete piling, steel piling, steel shells for cast-in-steel-shell concrete piling, and permanent steel casing for cast-in-drilled-hole concrete piling are typically listed in the SP[2] as being eligible for payment for *Materials on Hand but not yet incorporated in the work.*

Bar reinforcing steel used in cast-in-place concrete piling is typically listed in the SP[2] as being eligible for payment for *Materials on Hand but not yet incorporated in the work.*

Section 3-906E, *Materials on Hand*, of the *Construction Manual*, June 2013, states: "…In general, accept only completely fabricated units, ready for installation on the project with the following exceptions:

Piling—Steel plate used for steel pipe piling and driven steel shells filled with concrete and reinforcement as described in Section 49, "*Piling*," of the Standard Specifications may be considered acceptable as raw material. However, pay for such material as raw material only until shop fabrication of the pile is 100% complete. After shop fabrication is complete, the estimated fabricated value may be paid, subject to other specified restrictions and administrative guidelines."

Furnish and Drive

The following guidelines have been established to ensure uniform practice throughout the State for partial payments for piling. Refer to BCM 6-4.0, *Partial Payments*, for additional instructions regarding payment for Furnish Piling items.

- When steel or precast concrete piling of proper length are delivered to the job site ready for driving, the specification requirements for *furnishing* have been met and the material should be paid under *furnish piling* item on the progress pay estimate. Piles stored offsite, or onsite but not ready for driving, are to be considered as Materials-on-Hand.
- Portions of piling, such as steel shells for cast-in-place concrete piles, as described in Section 49-3 of the 2010 SS, are not complete piling and cannot be paid under the *furnishing* contract item. When the steel shells for cast-in-steel-shell concrete piles have

[2] 2010 SP, Section 9-1.16C or 2006 SP, Section 5.

been driven and the concrete and reinforcing steel have been placed to provide a complete pile, the contract item for *furnishing* may be paid.

- For steel pipe piling, full payment on the furnish item will not be made until the piling is on site and all field welds are completed and approved. This work includes welding of splices, and shear rings, when shown on the plans or required in the Special Provisions.
- The concrete filling material for cast-in-steel-shell concrete piling is paid under the furnish item while the placement of said material is paid under the drive item. This is particularly important when making item adjustments.
- For cast-in-drilled-hole concrete piling, permanent casing is paid as a separate item, but temporary casing is fully compensated in the piling item. Rock sockets are paid as a separate Cast-In-Drilled-Hole (Rock Socket) item.
- Bar reinforcing steel for cast-in-place concrete piling greater than or equal to 24 inches or 600 mm diameter is paid as a separate item. For smaller diameter cast-in-place concrete piling, bar reinforcing steel is included in the cast-in-place concrete piling item.
- For Cast-In-Drilled-Hole (CIDH) concrete piling constructed using the Wet Method (refer to BCM 130-7.0, *CIDH Concrete Piling*, for definition), payment for the CIDH concrete piling item and rock socket item (if applicable) will be made only after acceptance testing has been performed and the pile is accepted by the Engineer.

Pile Tip Revisions

The SS[3] specify how piles are paid for when the Engineer revises the pile tip.

Contracts using amended versions of the 2006 SS provide for payment for piling as follows:

> *When pile tips are revised by the Engineer for timber, steel, and precast prestressed concrete piles, and for cast-in-place concrete piles consisting of driven shells filled with concrete, the additional length required, including all materials, equipment, and labor for furnishing, splicing, and installing the piling, will be paid for as extra work as provide in Section 4-1.0D, "Extra Work".*

Contracts using the 2010 SS provide for payment for piling as follows:

> *If the Engineer revises the pile tip elevation for driven piles, the work involved in furnishing, splicing, and driving the additional length of pile is change order work.*

The length of piling that extends beyond the tip elevation shown on the plans, as ordered by the Engineer to meet design requirements, will be paid for as *Extra Work* or *change order work*.

[3] 2010 SS, Section 49-2.01C(1), *General*, and 2010 SS, Section 49-3.03C(1), *General*; or SS 2006, Section 49-6.02, *Payment*.

CIDH Concrete Piling

Cast-In-Drilled-Hole (CIDH) concrete piling can be grouped in two categories: the first is CIDH piling without inspection pipes (dry method), and the second is CIDH piles with inspection pipes (wet method).

The Standard Specifications (SS)[1] require inspection pipes and testing for all CIDH piling that are 24 inches (600 mm) in diameter or larger, except when the holes are dry or when the holes are dewatered without the use of temporary casing to control groundwater.

There has been much debate as to when the use of the dry method of pile construction is appropriate. Some have allowed the use of the dry method only when no water is encountered, while others have allowed the use of the dry method when a small amount of water is encountered. This inconsistent use of the dry method for construction of CIDH piles prompted Structure Construction (SC) to further investigate this matter. While performing its investigation, SC referenced Federal Highway Administration (FHWA) Publication No. FHWA – IF – 99 – 025, *Drilled Shafts: Construction Procedures and Design Methods* (excerpts shown in Attachment No. 1), which contain a commentary of the allowable use of the dry construction method when encountering water. Based on FHWA Publication No. FHWA – IF – 99 – 025, SC concludes that there are situations in which the use of the dry method for constructing CIDH piles when encountering small amounts of water is appropriate. The dry construction method will only be approved by the Engineer when the CIDH pile excavation demonstrates that less than 12 inches of water accumulates above the base over a one hour period without any pumping from the hole during this hour. Additionally and as a guideline for CIDH pile construction, three (3) inches of water in the bottom of the hole at the time of concrete placement will be acceptable if the pile design does not require end bearing. This must be verified through discussions with the Designer to gain a full understanding of the foundation design prior to the start of pile construction. Recently published FHWA NHI-10-016 *Drilled Shafts: Construction Procedures and LRFD Design Methods* reiterate these guidelines for using the dry construction method when a small amount of water is encountered. It should also be noted that the CIDH Pile Mitigation Committee Chairperson may be contacted regarding the appropriate use of the dry construction method. When encountering water during the construction of CIDH piles, there is no replacement for sound engineering judgment.

It is assumed that concrete placed in dry conditions is structurally sound. Piles less than 24 inches (600 mm) in diameter are designed assuming that the concrete will be placed in a dry or a dewatered hole. For these small diameter CIDH piles, if water is encountered and dewatering

[1] 2010 SS, Section 49-3.02A(4)(d)(i), *General*, or 2006 SP

does not work, immediately contact the Geotechnical Engineer or CIDH Pile Mitigation Committee Chairperson. The pile type may be inappropriate for the site conditions.

Concrete that is placed under slurry or placed in a dewatered hole using temporary casing must be inspected for quality. If there is soil contamination, slurry mixed with concrete, or zones of low density concrete, repairs might be required to make the concrete structurally sound.

Structure Construction publications provide extensive information regarding CIDH piling, and these publications are listed in the recommended reading order. Review of these publications is essential for administration of CIDH piling with inspection tubes.

1. *Foundation Manual*, Chapter 6, *Cast-in-Drilled-Hole Piles*:
   - This chapter provides general information on equipment and construction.

2. *Foundation Manual*, Chapter 9, *Slurry Displacement Piles*:
   - This chapter provides extensive details on the use of slurry. Read this chapter prior to reviewing the Contractor's CIDH pile placement plan.

3. *Foundation Manual*, Appendix K, Section K2, *Cast-in-Drilled-Hole (CIDH) Piles Construction Checklist*.

4. Bridge Construction Records and Procedures Manual, Bridge Construction Memos (BCM):

BCM 130-1.0	*Foundation Testing Branch*.
BCM 130-6.0	*Payment for Piling*.
BCM 130-7.0	*CIDH Concrete Piling*.
BCM 130-8.0	*CIDH Pile Mitigation Committee*.
BCM 130-9.0	*CIDH Pile Placement Plan and Concrete Test Batch*.
BCM 130-10.0	*Testing of CIDH Piling*.
BCM 130-11.0	*Simple Repair of CIDH Piling*.
BCM 130-12.0	*Mitigation of CIDH Piling*.
BCM 130-13.0	*CIDH Pile Information Submittal*.
BCM 130-14.0	*Slurry Test Kits for CIDH Piling*.
BCM 130-15.0	*Approved Synthetic Drilling Slurries*.
BCM 130-18.0	*Reduced Embedment Length of Column Reinforcement into Type II Shaft*.
BCM 130-20.0	*CIDH Pile Preconstruction Meeting*.
BCM 130-21.0	*CIDH Pile Non-Standard Mitigation Meeting*.

For technical assistance and contact information for the CIDH Pile Mitigation Committee Chairperson, see BCM 130-8.0.

A chronological outline for contract administration of CIDH piling *without inspection pipes* is shown below:

1. CIDH Pile Preconstruction Meeting:
   1.1. Conduct meeting per the Special Provisions and BCM 130-20.0.
2. Pile placement plan:

2.1. Review the plan.
2.2. Respond to the Contractor.

3. Pile Construction.

4. Payment.

A chronological outline for contract administration of CIDH piling *with inspection pipes* is shown below:

1. CIDH Pile Preconstruction Meeting:
   1.1. Conduct meeting per the Special Provisions and BCM 130-20.0.

2. Pile Placement Plan:
   2.1. Review the plan.
   2.2. Respond to the Contractor.
   2.3. Send a copy of the authorized placement plan to the CIDH Pile Mitigation Committee Chair.

3. Test batch of concrete:
   3.1. Witness the test.
   3.2. Review the results.
   3.3. Respond to the Contractor.
   3.4. For piles with inspection pipes, send a copy of the approved mix design and test results to the CIDH Pile Mitigation Committee Chair.

4. Pile Construction:
   4.1. Contractor logs concrete placement and submits copy within one working day.
   4.2. Contractor makes access for testing.
   4.3. Witness Contractor's probe of inspection pipes.
   4.4. Reject a pile if an inspection pipe is blocked.
   4.5. Notify the Foundation Testing Branch (FTB) of blocked inspection tubes and request guidance.(See BCM 130-10.0.)

5. Testing:
   5.1. Use the *CIDH Pile Acceptance Test Request Form*[2] to schedule testing with FTB so that testing can be completed as soon as possible.
   5.2. FTB performs tests and sends report.

6. Pile Acceptance or Rejection:
   6.1. Send a letter to the Contractor either accepting or rejecting a pile based on the FTB recommendations. For accepted piling, complete payment for that pile. Do not pay for rejected piling and continue with steps 7-13 below.

7. Suspend Depositing of Concrete (under slurry or with use of temporary casing to control groundwater):

[2] http://www.dot.ca.gov/hq/esc/geotech/ft/request.htm

7.1. Contractor submits revised pile placement plan to correct methods that resulted in anomalies.
7.2. Review revised pile placement plan.
7.3. Notify the Contractor when the plan is approved and slurry work can resume.

8. Pile Design Data Form:
   8.1. Immediately contact the Structures project design engineer, the project Geotechnical designer, and the corrosion specialist (Corrosion Technology Branch at Translab) to ensure that they complete the Pile Design Data Form (PDDF) included in the FTB report.
   8.2. Based on the completed PDDF, determine whether the rejected pile requires repair and if so, the feasibility of repairing the rejected pile. Consult with the CIDH Pile Mitigation Committee.
   8.3. Send a copy of the completed PDDF to the members in the CIDH Pile Mitigation Committee and allow 2 working days for a cursory check.
   8.4. Send appropriate letter and information to the Contractor (See BCM 130-10.0.)

9. CIDH Pile Non-Standard Mitigation Meeting. (This meeting is necessary only when a non-standard mitigation method is required, see BCM 130-12.0 for different mitigation methods.)
   9.1. Conduct meeting per the Special Provisions and BCM 130-21.0.

10. Pile Mitigation Plan:
   10.1. Contractor submits plan.
   10.2. Directly review if it is for simple repairs.
   10.3. Coordinate review with CIDH Pile Mitigation Committee for non-simple mitigation by sending a copy of the proposed Mitigation Plan to FTB and the CIDH Pile Mitigation Committee Chairperson.
   10.4. Get a consensus with CIDH Pile Mitigation Committee.
   10.5. Conduct a CIDH Pile Mitigation Review Meeting per BCM 130-12. This meeting is necessary only when a non-standard mitigation plan is rejected by the Designer.
   10.6. Review and respond to the Contractor until the plan can be approved.

11. Pile Mitigation:
   11.1 The Contractor submits the post mitigation report. Send a copy of this report to the members of the CIDH Pile Mitigation Committee.

12. CIDH Pile Information:
   12.1. Verify that all final data has been submitted on the CIDH Pile Information form to the CIDH Pile Mitigation Committee Chairperson.

13. Complete Payment.

Details for contract administration of CIDH piles are provided in BCM 130-8.0 to 130-21.0.

Volume II

CIDH PILE MITIGATION COMMITTEE

The CIDH Pile Mitigation Committee does the following:

- Provides technical support to Structure Representatives
- Reviews the Pile Design Data Form
- Reviews CIDH Pile Mitigation Plans to ensure statewide consistency for construction means and construction administration
- Ensure final acceptance of pile after repair mitigation work is completed.
- Liaisons with Industry
- Serves on Structure Design's Substructure Technical Committee
- Coordinates with METS, Corrosion Technology Branch

Members of the CIDH Pile Mitigation Committee consist of the CIDH Pile Committee Chair, the Foundation Testing Branch (FTB) Office Chief, the Project Engineer, and the Geotechnical Designer.

Current contact information for the CIDH Pile Mitigation Committee Chair is identified on the Offices of Structure Construction link at: http://onramp.dot.ca.gov/hq/oscnet/sc_people/hq_people.htm#os3.

Mail: Caltrans Courier Mail:
Engineering Services, OSC
P.O. Box 168041, Mail Station 9-2 11H 1801 30th Street, Mail Station 9-2/11H
Sacramento, CA 95816-8041

Current contact information for the Foundation Testing Branch office chief is identified on the Office of Geotechnical Support link at: http://www.dot.ca.gov/hq/esc/geotech/geo_support/geo_support.html.

Mail: Caltrans
Transportation Laboratory, Geotechnical Services
5900 Folsom Blvd, Mail Station 5
Sacramento, CA 95819-4612

CIDH Pile Installation Plan and Concrete Test Branch

All Cast-In-Drilled-Hole (CIDH) Piling
The Contractor is required to submit a Pile Installation Plan for all CIDH piling. A listing of the plan requirements is in the Standard Specifications (SS) and Special Provisions (SP)[1]; concrete mix design is one of the items. Reject the Pile Installation Plan if it is not complete, not appropriate, or not adequate for the site conditions and project requirements. Notify the Contractor in writing when the Pile Installation Plan is authorized.

CIDH Piling with Inspection Pipes and Testing
When inspection pipes and testing are required, additional items are required and these are listed in the specifications[1].

The specifications[2] require a concrete test batch for concrete to be placed under slurry. The Contractor must submit the Pile Installation Plan prior to producing the test batch, and at least 15 days prior to constructing piling. Verify that the placement time is consistent with the proposed mix design. Potential issues that should be addressed for placement of concrete under slurry are detailed in the *Foundation Manual*, Chapter 9, *Slurry Displacement Piles*.

Witness the Contractor's testing of a batch of concrete. Receive the test batch results and review for performance, as well as consistency with the Pile Installation Plan. If the performance of the test batch and the Pile Installation Plan do not match, then reject the plan. The Contractor is required to revise the Pile Installation Plan (including the mix design) until the plan and the test batch results are consistent. If the mix design requires revision, then another testing of a batch of concrete is required.

For piling with inspection pipes and testing, send a copy of the authorized Pile Installation Plan and authorized mix design (including the approved test batch results) to the CIDH Pile Committee Chairperson.

[1] 2010 SS, Section 49-3.02A(3)(b), *Pile Installation Plan*, or 2006 SP
[2] SS 2010, Section 49-3.02A(4)(c), *Concrete Test Batch*. or 2006 SP

Testing of CIDH Piling

Testing

The Foundation Testing Branch (FTB), located in Sacramento, provides statewide foundation testing services. The FTB's workload fluctuates widely throughout the year; therefore, in order to provide timely services, advance notice will be required for scheduling Gamma-Gamma testing, especially at the beginning of a new contract. Notify FTB thirty (30) days before testing is needed, or as early as possible. Obtain the current version of the *CIDH Pile Acceptance Test Request* Form from FTB's website[1].

Complete the form, providing an estimated date for testing, and fax the form to the FTB number provided on the form. The FTB will assign an engineer who will contact you for further scheduling and foundation testing on your project.

Coordinate the Contractor's pile construction operations with FTB. Notify FTB so piling can be tested as soon as possible. Remember, Gamma-Gamma testing can be performed even before the concrete is cured. It is important to inform FTB as soon as possible so the Contractor does not construct numerous CIDH piles before FTB can perform testing. The goal is to avoid having to reject several piles with the same problem. If the Contractor's construction methods do not work, the Contractor needs to correct the methods prior to depositing any additional concrete for Cast-In-Drilled-Hole (CIDH) piles with inspection tubes.

In order for FTB to perform Gamma-Gamma testing, inspection pipes should be completely accessible for the Gamma-Gamma probe and free of water (Contractors will typically fill tubes with water during construction. They should be purged prior to testing.) The Contractor checks the inspection pipes for accessibility by passing a probe (a 1-1/4 inch diameter by 4-1/2 feet long rigid cylinder) through the length of the pipe. The Engineer must witness the entire probe check of the inspection pipes. When the inspection pipes are confirmed to be clear, immediately notify FTB using the *CIDH Pile Acceptance Test Request* Form so that testing can be performed. Ensure that the Contractor has provided access to the pile for the FTB Engineer. If an inspection pipe is blocked, this is considered to be an anomaly and the pile will immediately be rejected. The Contractor can *core* a hole to mitigate a blocked pipe. Coring must be performed in accordance with the requirements of the Standard Specifications (SS) and Special Provisions (SP).[2] The Contractor logs the coring operation and provides the cored materials to the Engineer. Send a copy of the coring report to the FTB for review and evaluation of the portion of the pile represented by coring. No Gamma-Gamma testing is performed in the cored holes. Although the

[1] www.dot.ca.gov/hq/esc/geotech/requests/cidh.pdf
[2] 2010 SS, Section 49-3.02A(4)(d)(ii), *Vertical Inspection Pipes*, or 2006 SP.

specifications require coring to mitigate a blocked tube, there are cases where coring may not be necessary. For example, if the blockage is within the bottom one pile diameter, and the pile does not require end bearing, then it is likely that the pile is adequate from a structural and geotechnical standpoint without verification of the concrete condition below the blocked zone. Therefore, it is recommended to contact FTB or the CIDH Pile Mitigation Committee Chair for guidance prior to coring for blocked tubes.

The Foundation Testing Branch will perform testing and submit a *Pile Acceptance Test Report*. The FTB will transmit the report via mail and e-mail to the Structure Representative (SR), Structure Construction Headquarters (SC HQ), Structure Design, Geotechnical Services, and the Corrosion Engineer. In order to expedite the process, especially when the pile is rejected, FTB may also e-mail the *Pile Acceptance Test Report* directly to the General Contractor and Drilling Subcontractor. This should be determined during the CIDH Pile Pre-Construction meeting (refer to Bridge Construction Memo (BCM) 130-20.0, *CIDH Pile Preconstruction Meeting.*) The SR should verify during the meeting whether *FTB Acceptance Test Report* will be e-mailed simultaneously to the General Contractor and Drilling Subcontractor.

If the pile is free of anomalies, FTB will recommend pile acceptance. If the pile has an anomaly, the location and details will be provided, and FTB will recommend rejection. Follow the recommendation in the FTB report and immediately notify the Contractor of either pile acceptance or rejection. A sample rejection letter is shown in Attachment No. 1.

Rejected Piling

If an anomaly is found, the pile is rejected. An anomaly may be due to soil contamination, a zone of low-density concrete, or slurry mixed with concrete. An anomaly may or may not represent a defect in the CIDH pile. Therefore, each anomaly must be investigated separately.

In some cases, FTB will release a *Gamma-Gamma Logging Acceptance Test Report* and then might choose to do additional testing (such as cross-hole sonic logging) to better define the type and limits of an anomaly. When FTB plans to do additional testing, do not wait on these results; send the rejection letter. The FTB decision whether to perform additional tests will be evaluated and presented in the Gamma-Gamma report.

When a pile is rejected, suspend all depositing of concrete under slurry (or suspend using temporary casing to control groundwater) until the Contractor submits a revised pile placement plan. The revised plan must explain how the anomaly occurred and what changes are made to avoid the same problem. Immediately notify the Contractor when a revised pile placement plan has been reviewed, the new plan is acceptable, and the work may resume.

In the pile test report, FTB will include the *Pile Design Data* form. A sample form is shown in Attachment No. 2. Completion of this form requires input from the project Structural Designer, the project Geotechnical Designer and the Corrosion Specialist (contact Corrosion Technology Branch at Translab). Do not allow excessive delays to occur in completing this form. If one or more of the responsible persons are unable or unwilling to provide the data needed to complete this form in a timely manner, immediately elevate this to the CIDH Pile Committee Chairperson. After completing this form submit it to the CIDH Pile Mitigation Committee and allow at least

two working days for review. Discuss the pile design requirements with the CIDH Pile Mitigation Committee to determine whether the rejected pile requires mitigation, and if so, to determine viable mitigation methods (refer to BCM 130-12.0, *Mitigation of CIDH Piling*). A pile with multiple anomalies may have multiple *Pile Design Data* forms.

Based on the *Pile Design Data* information and the discussion of whether the rejected pile requires mitigation, proceed with one of the following actions:

1. Determine that the anomaly does not affect the necessary design performance and that the anomaly does not affect the necessary corrosion resistance, so mitigation is not required (consensus with the CIDH Pile Mitigation Committee is required). The Contractor can forego mitigation that is not required and accept an administrative deduction or mitigate the pile for full payment. Notify the Contractor in writing. A sample letter is shown in Attachment No. 3.

2. Determine that a *simple repair* can be used. See BCM 130-11.0, *Simple Repair of CIDH Piling*. Notify the Contractor in writing. A sample letter is shown in Attachment No. 4.

3. Determine that the anomaly must be mitigated and evaluate viable mitigation methods (refer to BCM 130-12.0, *Mitigation of CIDH Piling*). Notify the Contractor in writing. A sample letter is shown in Attachment No. 4.

4. For anomalies that require a non-standard mitigation plan, the Contractor is required to hold a CIDH Pile Non-Standard Mitigation Meeting per the Special Provisions (refer to BCM 130-21.0, *CIDH Pile Non-Standard Mitigation Meeting*). Notify the Contractor in writing. A sample letter is shown in Attachment No. 4.

DEPARTMENT OF TRANSPORTATION
<Your Office Address>
<Your Office Phone>

Serious Drought.
Help save water!

Month date, year

File: <Project Name>
<Co/Rte./Pm>
<Job EA>

<Contractor Name>
<Contractor Address>

Dear <Responsible Person>,

The attached CIDH pile acceptance test report for piles <pile numbers>, dated <report date>, has indicated the presence of anomalies in pile <pile number>, located at Bridge No. xx-xxxx, <Bridge Name>. Pile <rejected pile number> is hereby rejected in accordance with the specifications[1].

You are reminded of your responsibilities in the specifications[2], which require "…written changes to the methods of pile construction…" before concrete placement in the remaining piles, with inspection pipes, can continue. No concrete placement for these piles will be allowed until your revised pile installation plan has been received and authorized by the Engineer.

An investigation is being performed to determine whether mitigation of pile <pile number> is required, and if so, whether pile <pile number> can be repaired or must be supplemented or replaced. You will be notified of the results of this investigation as soon as it has been completed.

Edit As Appropriate

As indicated in the Gamma-Gamma Logging Acceptance Test Report, the Foundation Testing Branch will (will not) perform additional testing to further evaluate the rejected pile. You may perform your own testing on the rejected pile.

Sincerely,

Resident Engineer
Attachments <Pile Acceptance Test Report

[1]2010 SS, Section 5-1.30, *Noncompliant and Unauthorized Work*, or 2006 SS, Section 5-1.09.
[2]2010 SS, Section 49-3.02A(4)(d)(iv), *Rejected Piles*, or 2006 SP

Sample Pile Design Data Form

1 Foundation Testing

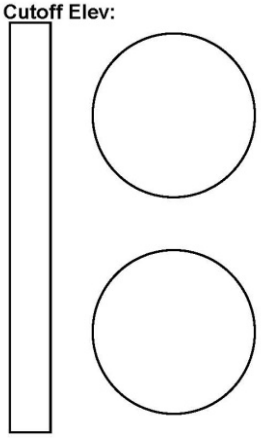
Name:
Phone:
Date:

Anomaly Overview

Testing Performed ☐ GGL ☐ CSL

Shaft Diameter:

Cutoff Elev:



Tip Elev.:

Anomaly Description

Section A-A: _____

_____

_____

Section B-B: _____

_____

_____

2 Geotechnical

Name:
Phone:
Date:

Required Nominal Resistance of Shaft (per contract plans)

Compression:_____kips Tension:_____kips

Lowest Estimated Groundwater Elevation: _____

Remaining Required Nominal Resistance To Be Developed Below Each Anomalous Section:

Section A-A: Compression_____Tension_____kips

Soil and/or Rock Type: _____

Shaft is geotechnically ☐ Acceptable ☐ Unacceptable

Section B-B: Compression_____Tension_____kips

Soil and/or Rock Type: _____

Shaft is geotechnically ☐ Acceptable ☐ Unacceptable

Comments:_____

_____

3 Structural

Name:
Phone:
Date:

As-Designed Capacity of Shaft

Section A-A: Shear: _____ Moment: _____

Section B-B: Shear: _____ Moment: _____

Maximum Demand of Shaft at Section A-A

Shear: _____ Moment: _____

Shaft is structurally ☐ Acceptable ☐ Unacceptable

Maximum Demand of Shaft at Section B-B

Shear: _____ Moment: _____

Shaft is structurally ☐ Acceptable ☐ Unacceptable

Comments:_____

_____

4 Corrosion

Name:
Phone: Date:

Consideration is ☐ Required ☐ Not required

For anomalies between the top of pile and 3 feet below the lowest estimated groundwater level at the site, corrosion results listed in the Geotechnical report are used to assess the need for repair. For situations where results are not available, soil samples may be obtained adjacent to the anomaly and tested in accordance with California Test (CT) 643 (Parts 2, 3 and 4) and if necessary, CT 417 and CT 422 to determine soil corrosivity. For anomalies outside these limits, and where no stray current source can be identified, or for non-corrosive soil conditions, no consideration of corrosion potential is required.

Corrosion Potential at Section A-A: _____

Corrosion Potential at Section B-B: _____

5 Construction

Considering parts 2-4 of this form,

Structure Rep.:
Phone: Date:

Sec. A-A is: ☐ Acceptable with Administrative Deduction ☐ Unacceptable, Mitigation is Required

Sec. B-B is: ☐ Acceptable with Administrative Deduction ☐ Unacceptable, Mitigation is Required

Bridge Name:	Bridge No.:	Abut/Bent:
Dist-Co.-Rte:	EA:	Pile:
Structure Rep.:	Phone:	Fax:

DEPARTMENT OF TRANSPORTATION
<Your Office Address>
<Your Office Phone>

Serious Drought.
Help save water!

Month date, year

File: <Project Name>
<Co/Rte./Pm>
<Job EA>

<Contractor Name>
<Contractor Address>

Dear : <Responsible Person>,

Please refer to my letter dated <letter date> regarding the rejection of pile <pile number>, located at Bridge No. xx-xxxx.

An investigation of anomaly(ies) was performed by the Engineer and it was determined that mitigation work is not required. If you elect not to mitigate the anomaly(ies), payment will be reduced for the anomaly(ies) in conformance with the specifications [1]. Full payment will be made if mitigation is completed and accepted.

Please inform me of your decision to either mitigate or take a reduction in payment.

Sincerely,

Resident Engineer

[1] 2010 SS, Section 49-3.02A(4)(d)(iv), *Rejected Piles*, or 2006 SP

DEPARTMENT OF TRANSPORTATION
<Your Office Address>
<Your Office Phone>

Serious Drought.
Help save water!

Month date, year

File: <Project Name>
<Co/Rte./Pm>
<Job EA>

<Contractor Name>
<Contractor Address>

Dear <Responsible Person>,

Please refer to my letter dated <letter date> regarding the rejection of pile <pile number>, located at Bridge No. xx-xxxx.

An investigation of the rejected pile performed by the Engineer has determined that mitigation work is required.

Edit As Appropriate

Action 3:
You are reminded of your responsibilities in the specifications[1], which require "….a plan for repair, removal, or replacement of the rejected piling" before the rejected pile can be accepted.

Action 4:
You are reminded of your responsibilities in the specifications[1], which require "…..schedule and hold a CIDH Pile Non-Standard Mitigation Meeting within five business days after the Engineer's determination whether the rejected pile requires mitigation".

Attached is a copy of the original pile acceptance test report, < the cross-hole sonic pile test report if available>, and the pile design requirements to aid you in the preparation of the pile mitigation plan.

Please submit a pile mitigation plan to this office for review and approval as soon as possible.

Sincerely,

Resident Engineer

Attachments <Pile Acceptance Test Reports, Pile Design Data form>

[1] 2010 SS, Section 49-3.02(A)(d)(iv), *Rejected Piles*, or 2006 SP.

Simple Repair of CIDH Piling

A *Simple repair* is allowed for piling with the following conditions:

- The anomaly is within 5 feet of the top of the pile.
- There are no other repairs in the same pile.
- The repair area can be made completely visible.

Simple repairs consist of excavation of soils and then removal and replacement of defective concrete. Excavation of soils below five feet will reduce the geotechnical capacity of the pile and is not allowed. If the Contractor wants to excavate more than five feet from the top of the pile, this is not a simple repair and the process in BCM 130-12, *Mitigation of CIDH Piling*, must be used.

If permanent casing is within five feet of the top of the pile, it might not be feasible to make the defective concrete completely visible, and this is not a simple repair. The CIDH Pile Mitigation Committee must be consulted when an anomaly is inside the permanent casing.

Prior to starting simple repairs, the Contractor is required to submit the pile mitigation plan. Although the CIDH Pile Mitigation Committee is available for consultation on simple repairs, it is not required to send the mitigation plan to the CIDH Pile Mitigation Committee for review and consensus.

After the simple repairs are completed and approved, submit the CIDH Pile Information (see BCM 130-13.0, *CIDH Pile Information Submittal*), along with the Contractor's post mitigation report, to the CIDH Pile Mitigation Committee Chair.

Mitigation of CIDH Piling

A rejected pile is only required to be mitigated to the extent needed for the pile to perform as intended by the design requirements. The CIDH Pile Mitigation Committee[1] will provide technical support to the Structure Representative, and assist in the review of submittals for repair, so that mitigation work is appropriate for the design, and administered consistently statewide. Common anomalies and mitigation measures are included in the *Foundation Manual*, Chapter 9, *Slurry Displacement Piles*. If the Contractor proposes to do the anomaly investigation, immediately consult with the CIDH Pile Mitigation Committee.

As presented in Bridge Construction Memo (BCM) 130-10.0, the *Pile Design Data Form* will indicate whether the rejected pile requires mitigation and facilitates determining the feasibility of using a grouting repair method to mitigate the pile. Grouting repair cannot be expected to restore cross sections in zones of high moment demand.

Mitigation of a defective CIDH pile can be grouped into four methodologies:

<u>Standard Repair Methodology:</u>

1. Unearth and Recast (Basic Repair).
2. Pressure Grout (Grouting Repair).

<u>Non-Standard Repair Methodology:</u>

3. Structural Bridging.
4. Replacement/Supplement.

The first two methods are considered *standard repair methods* and are covered by the Association of Drilled Shaft Contractors (ADSC) Standard Mitigation Plan. *Standard Mitigation Plan* refers to the fact that the plan is of an established procedure; it does not endorse that method to address a particular anomaly repair. The latest version of this plan can be accessed through the Foundation Testing Branch website at[2].

If it is feasible, an anomaly can be mitigated with repairs (basic, grouting, structural bridging). If it is not feasible to repair the anomaly, then the pile has to be replaced or supplemented with additional piling. The repair strategy is at the option of the Contractor, and subject to Caltrans approval. No additional payment is made for any type of mitigation of rejected piling.

For anomalies that require a non-standard mitigation plan (i.e. basic repair or grouting repair are not feasible/acceptable), the Contractor is required to schedule a CIDH Pile Non-Standard

[1] BCM 130-8.0, *CIDH Pile Mitigation Committee*.
[2] http://www.dot.ca.gov/hq/esc/geotech/ft/adscmitplan.htm

Mitigation Meeting, per the specifications,[3] to address a viable non-standard mitigation plan (structural bridging and replacement/supplement) in a timely manner. BCM 130-21, *CIDH Pile Non-Standard Mitigation Meeting,* provides guidance for conducting this meeting.

The Contractor must have a pile mitigation plan submitted to the Structure Representative, and the Structure Representative must approve it before the mitigation work begins. A list of the plan requirements is in the specifications[4]. When a Contractor selects the ADSC Standard Mitigation Plan, all applicable contractual elements of the mitigation plan, as presented in the specifications, need to accompany the ADSC Standard Mitigation Plan.

Review the mitigation plan to ensure it is complete. Call the CIDH Pile Mitigation Committee Chair if there are questions. Send copies of the plan to the CIDH Pile Mitigation Committee. The mitigation plan will be reviewed by the Foundation Testing Branch. Consensus from the Committee is required before the plan can be accepted. The Committee Chair will send the consensus with written recommendations to the Structure Representative.

Notify the Contractor if the plan is rejected (typically due to insufficient detail or inappropriate procedures). A sample letter for approval of the pile mitigation plan submitted to the Contractor is shown in Attachment No. 1.

If a non-standard mitigation plan is rejected by the structural or geotechnical designer, schedule and hold a Pile Mitigation Plan Review Meeting between the project designer and the mitigation plan designer. The purpose of this meeting is to clarify deficiencies of the proposed mitigation plan, through direct communication between the mitigation plan designer and reviewer, in order to expedite the re-submittal and approval of the mitigation plan.

Generally, the pile can be accepted if the mitigation work is performed in accordance with the provisions of the approved pile mitigation plan. However, there are circumstances when the pile must be retested. Acceptance criteria of a mitigated pile (i.e. retesting, coring) will be provided in the mitigation plan review report. The acceptance criteria must be included in the pile mitigation plan approval letter (Attachment No. 1).

After the repair, supplemental, or replacement work is complete and approved, the Contractor submits a post mitigation report per the specifications[5]. Send a copy of the post mitigation report to the CIDH Pile Mitigation Committee. Upon completion of the piling work, submit the CIDH Pile Information (see BCM 130-13.0) to the CIDH Pile Mitigation Committee Chair.

[3] 2010 SS, Section 49-3.02A(4)(d)(iv), *Rejected Piles*, or 2006 SP.
[4] 2010 SS, Section 49-3.02A(3)(g), *Mitigation Plans*, or 2006 SP.
[5] 2010 SS, Section 49-3.02A(3)(h), *Mitigation Report*, or 2006 SP.

DEPARTMENT OF TRANSPORTATION
<Your Office Address>
<Your Office Phone>

Serious Drought.
Help save water!

Month day, year

File: <Project Name>
<Co/Rte./Pm>
<Job EA>

<Contractor Name>
<Contractor Address>

Dear : <Responsible Person>,

The CIDH pile mitigation plan, dated <date>, submitted for pile <abutment/bent number, pile number> at the <bridge name, bridge number> has been reviewed and is satisfactory.

<CONTINGENCIES PARAGRAPH- If the mitigation plan is approved. Contingent upon anything, list it here.>

<CRITERIA FOR PILE ACCEPTANCE PARAGRAPH- Certain criteria might be required after the mitigation work is completed to show the mitigation was successful (i.e. additional Gamma-Gamma testing, Cross-hole Sonic Logging or coring). List it here.>

You are reminded of your responsibilities[1], which require: *For each rejected pile, submit a mitigation plan for repair, supplementation, or replacement. The mitigation plan must....*

Sincerely,

Resident Engineer

[1] 2010 SS, Section 49-3.02A(3)(g),*Mitigation Plans*, or 2006 SP

CIDH Pile Information Submittal

Structure Construction maintains a database on CIDH pilings that have been tested for anomalies. After a test report from the Foundation Testing Branch (FTB) is sent to the Structure Representative, the CIDH Pile Mitigation Committee Chair will send an electronic copy of *CIDH Pile Informatio*n form (Attachment No. 1) to the Structure Representative.

When all of the CIDH piles are complete for a given contract, the Structure Representative will make one submittal of the final results to the CIDH Pile Mitigation Committee Chair. A sample of a completed form is shown in Attachment No. 2.

If there are piles for a contract that require mitigation, wait to complete this form until after mitigation is completed and the piles are accepted.

State of California
DEPARTMENT OF TRANSPORTATION

California State Transportation Agency

Memorandum

Serious drought.
Help save water!

To: ROBERT A. STOTT
DEPUTY DIVISION CHIEF
STRUCTURE CONSTRUCTION
DIVISION OF ENGINEERING SERVICES, MS 9-2/11H

Date: <Month Day, Year>

ATTENTION: CHAIRMAN – PILE MITIGATION COMMITTEE

From: <NAME>
Structure Representative
<Field Office Address>
<City, CA Zip >
<Phone #>

Subject: CIDH Pile Information for Piles Tested by the Foundation Testing Branch (FTB)

Project No.:	Project Name/ Location:	
Bridge/Structure No.:	Bridge/Structure Name:	
Abut/Bent/RW/SW No.:	Prime Contractor:	
Const. Phase: *(if applicable)*	Drilling Contractor:	
Slurry Type: *(water, SlurryPro, etc)*	Concrete Mfr & Mix Design No.:	Max. Concrete Aggregate Size:

PILE No.	From Acceptance Test Report, PILE was ACCEPTED *(A)* OR REJECTED *(R)*?	IF Pile was REJECTED, List ANOMALY X-Sections *(A-A,B-B,etc)*	FOR ANOMALY X-SECTIONS		
			Was MITIGATION REQUIRED? *(Yes/No)*	IF Mitigation Required, was Mitigation DUE to GEOTECH *(G)*, STRUCTURE DESIGN *(S)*, and/or CORROSION *(C)* concerns?	Describe Repair Method *(simple repair, excavate/chip/concrete, waterblast/grout, etc)* OR Amount of Administrative Deduction Taken *($$)*

See BCM 130-13.0, Attachment 2, for an example of a completed form. Sheet _____ of _____ pages

"Provide a safe, sustainable, integrated and efficient transportation system to enhance California's economy and livability"

State of California
DEPARTMENT OF TRANSPORTATION

California State Transportation Agency

Memorandum

Serious drought.
Help save Water!

To: ROBERT A. STOTT
DEPUTY DIVISION CHIEF
STRUCTURE CONSTRUCTION
DIVISION OF ENGINEERING SERVICES, MS 9-2/11H

Date: 6/30/2014

ATTENTION: CHAIRMAN – PILE MITIGATION COMMITTEE

SAMPLE

From: BASSEM KABBARA
Structure Representative
1129 S. Woodruff Ave.
Downey, CA 90241
(562) 401-3333

Subject: CIDH Pile Information for Piles Tested by the Foundation Testing Branch (FTB)

Project No.:	*07-1199U4*	Project Name/ Location:	*101/Eastside Light-Rail Underpass*		
Bridge/Structure No.:	*53-2975*	Bridge/Structure Name:	*Eastside Underpass (LRT)*		
Abut/Bent/RW/SW No.:	*Abut. #1*	Prime Contractor:	*Brutoco Eng. & Const.*		
Const. Phase: *(if applicable)*	*N/A*	Drilling Contractor:	*Malcolm Drilling Co.*		
Slurry Type: *(water, SlurryPro, etc)*	*KB SlurryPro CDP*	Concrete Mfr & Mix Design No.:	*National; 04-SE8805R*	Max. Concrete Aggregate Size:	*3/8"*

PILE No.	From Acceptance Test Report, PILE was ACCEPTED *(A)* OR REJECTED *(R)*?	IF Pile was REJECTED, List ANOMALY X-Sections *(A-A,B-B,etc)*	FOR ANOMALY X-SECTIONS		
			Was MITIGATION REQUIRED? *(Yes/No)*	IF Mitigation Required, was Mitigation DUE to GEOTECH *(G)*, STRUCTURE DESIGN *(S)*, and/or CORROSION *(C)* concerns?	Describe Repair Method *(simple repair, excavate/chip/concrete, waterblast/grout, etc)* OR Amount of Administrative Deduction Taken *($$)*
1	*R*	*A-A*	*Y*	*G, S, C*	Simple Repair
		B-B	*Y*	*G, S, C*	Water Jet/Permeation Grouting
1		*C-C*	*Y*	*G, S*	Core drill center, replace w/conc.
1		*D-D*	*N*	*-*	*$400*
2	*R*	*A-A*	*Y*	*G, S, C*	*Simple Repair*
2		*B-B*	*N*	*-*	*$400*
3	*R*	*A-A*	*Y*	*S*	*Simple Repair*
3		*B-B*	*Y*	*G, S*	Water Jet/Permeation Grouting
3		*C-C*	*N*	*-*	Water Jet/Permeation Grouting
3		*D-D*	*N*	*-*	*$805*
4	*R*	*A-A*	*N*	*-*	*$400*
4		*B-B*	*N*	*-*	*$1635*
5	*A*	*-*	*-*	*-*	*-*
6	*A*	*-*	*-*	*-*	*-*

Sheet 1 of 1 pages

"Provide a safe, sustainable, integrated and efficient transportation system to enhance California's economy and livability"

Slurry Test Kits for CIDH Piling

Structure Construction (SC) has slurry test kits for use in the field. Witness the Contractor's quality control testing of slurry and then randomly perform quality assurance testing using the kit. If your field office does not have a kit, contact your Bridge Construction Engineer.

The slurry test kits include equipment for testing density, sand content, marsh funnel viscosity, and pH. The test procedures for density, marsh funnel viscosity, and sand content can be accessed through the SC Intranet web site[1].

The requirements for the Contractor's testing of slurry are in the Special Provisions. Extensive background and commentary on slurry (including the reasons for testing slurry) are in the *Foundation Manual*, Chapter 9, *Slurry Displacement Piles*.

Key inspection considerations are covered in the *Foundation Manual* and are summarized as follows:

- Before the slurry operation starts, verify that the Contractor's sampling and testing equipment is adequate.
- Make sure the Contractor is proportioning, mixing, agitating (or circulating) per the specifications. Sand can quickly settle out, especially with synthetic slurries. Mixing or agitating ensures accurate test results.
- Make sure the Contractor samples at the correct elevations.
- Slurry test results from the same hole (but different elevations) may vary, but each test result must conform to the specifications.

For synthetic slurry (not mineral slurries), occasionally take a sample and send it in to Translab to verify that the chemistry still matches the pre-approved product chemistry. On the sample testing ticket request that a copy of the results be sent to the CIDH Pile Committee Chair (see BCM 130-8). Send about 8 ounces (200 ml) of the mixed slurry to:

Caltrans
Transportation Laboratory, Chemical Testing Branch
5900 Folsom Boulevard, Mail Station 5
Sacramento, CA 95819-4612

[1] Using the tab, *Field Resources/ASTM AWS Specs*, search for (APIRP 13B-1) which will take you to a link. http://onramp.dot.ca.gov/hq/oscnet/

Volume II

APPROVED SYNTHETIC DRILLING SLURRIES

The following synthetic drilling slurries have been approved for Cast-In-Drilled-Hole (CIDH) piles:

PRODUCT M	ANUFACTURER
SlurryPro CDP	KB International, LLC 735 Broad Street, Suite 209 Chattanooga, TN 37402 (423) 266-6964
Super Mud	PDS Company 105 West Sharp Street El Dorado, AR 71730 (800) 243-7455
Shore Pac GCV	CETCO Drilling Products Group 2870 Forbs Avenue Hoffman Estates, IL 60192 (800) 527-9948
Terragel or Novagel Polymer	Geo-Tech Drilling Fluids 220 N. Zapata Hwy, Suite 11A Laredo, TX 78043 (210) 587-4758

Verify that the synthetic slurry is appropriate (per the manufacturer's recommendations) for the site geology. Do not allow synthetic slurry in primarily soft or very soft cohesive soils. When in doubt, consult with the person that signed the Foundation Recommendations for the project.

Testing requirements for each of the slurries are listed in the Special Provisions. For the most current synthetic slurry specifications see Standard Specifications[1] (SS) and the Standard Special Provisions[2] (SSP).

If the Contractor proposes to use slurry admixtures, discuss the type, concentration and mixing with the CIDH Pile Committee Chairperson. Questions regarding the use of slurries can be directed to the CIDH Pile Committee Chairperson.

[1] 2010 SS, Section 49-3.02 B (6)(c)
[2] 2010 SSP, Section 49-3.02 B (6)(c) or 2006 SSP, Section 49-311.

Per Section 49-3-02 B (6)(c) of the 2010 Standard Specification, a manufacturer's representative must:

1. Provide technical assistance for the use of the material.
2. Be at the job site before introduction of the synthetic slurry into the drilled hole.
3. Remain at the job site until released by the Engineer.

Settlement Platforms

Many projects that include construction of new embankments and surcharge embankments will require minimum settlement periods. Check the Special Provisions (SP) for the specified settlement periods. Review the Geotechnical Design Report (GDR) or Final Foundation Report (FFR) for foundation recommendations, additional settlement information, and to identify a contact person at Geotechnical Services.

Typically, the anticipated total settlement will be no more than a couple of inches and may be verified by the installation and use of temporary survey hubs. Make sure the Contractor protects the hubs during the period of monitoring. Common settlement periods are either 30 or 60 days, but special cases may require longer time periods.

Occasionally the anticipated total embankment settlement may exceed several inches, and in some instances may be more than one foot. In such cases, the plans and special provisions should include requirements for the installation and monitoring of settlement platforms. These requirements are site specific and custom developed for each contract. In addition to the settlement platforms, limits for maximum rate of fill placement, surcharge heights, wick drains, piezometers, and slope inclinometers may also be specified in these cases. After the contract is awarded, notify Geotechnical Services to begin coordination of the settlement platform construction and monitoring.

Review the *Settlement Periods and Surcharges* section of the Standard Specifications[1]. Review the special provisions and plans for each contract. In some cases, the contract may require the Contractor to install the settlement platforms and to do the monitoring. In other cases, the contract may require the Contractor to allow time and provide access so that Geotechnical Services can install the platforms and do the monitoring. In this case, careful coordination is necessary in order to avoid delays.

Information on the method of installation and use of embankment settlement devices can be found in *California Test Methods (CTM)*, California Test 112. Ensure that the Contractor does not disturb the platforms during the monitoring period. If the platform is damaged, it must be repaired or replaced in a timely manner.

Discuss the results of the settlement monitoring on a regular basis with Geotechnical Services. Obtain recommendations from Geotechnical Services on the progress of settlement and notification of when the settlement is complete. The Engineer may order an increase or decrease of any estimated settlement period. This may increase or decrease the number of working days

[1] 2010 SS, Section 19-6.03D, *Settlement Periods and Surcharges*, or SS 2006, Section 19-6.025, *Settlement Period*.

allowed for the completion of work if the settlement period is considered to be the current controlling operation. Typically, the specification for increasing or decreasing the duration of the required settlement period is written in the *Earthwork* section of the Special Provisions. Neither the Contractor nor Caltrans will be entitled to any compensation other than an adjustment of contract time due to increases or decreases in the settlement periods.

Reference to the Foundation Manual–Appendix K for Construction Checklists for Foundation Work

Construction checklists for the following foundation work are located in Appendix K of the *Foundation Manual* :

- Driven Pile.
- Cast-in-Drilled-Hole (CIDH) Pile.
- Cofferdam Seal-Course.
- Footing Foundation.
- Micropile.
- Ground Anchor.
- Soil Nail Wall.

These checklists have been developed to assist field personnel in preparing documents and inspecting fieldwork to ensure compliance with contract requirements.

The checklists can be downloaded from the Structure Construction (SC) Intranet link[1].

For questions, contact the SC Substructure Technical Team. A list of the team members is available at the SC Intranet link[2].

[1] http://onramp.dot.ca.gov/hq/oscnet/sc_manuals/foundation/
[2] http://onramp.dot.ca.gov/hq/oscnet/osc_tech_com/04-Substructure/BCE_Substructure.htm

Volume II

CAST-IN-DRILLED-HOLE (CIDH) PILE PRECONSTRUCTION MEETING

For all projects advertised after January 2012 (and in some cases by addendum) a section titled "CIDH Pile Preconstruction Meeting" should be added to the 2010 Standard Specifications[1].

The purpose of the Preconstruction CIDH Pile Meeting is to establish contacts and communication protocol for the Contractor, the Engineer and their representatives involved in CIDH pile design and construction, and to afford all parties a common understanding of the construction process, acceptance testing, and mitigation of CIDH piles. Attendance for this meeting is mandatory for the Resident Engineer, Structure Representative, Assistant Structure Representatives, Foundation Testing Branch and CIDH Pile Mitigation Committee representatives, Structural and Geotechnical Designers that are providing construction support for the project, Contractor's Project Manager, Drilling Subcontractor's Project Manager, Project Superintendent, Drilling Subcontractor's Superintendent/Foreman, and the Reinforcing Steel Subcontractor's Foreman/Superintendent.

Bridge Construction Memo (BCM) 130-20.0, Attachment Number 1 is a general meeting agenda to assist you with understanding the steps involved in construction, acceptance testing and mitigation of CIDH piles. However, bear in mind that these are reminders only. Review the general meeting agenda with regard to your specific project and modify the meeting agenda as necessary. Certain steps may or may not be included depending upon their applicability to a specific project.

[1] SS 2010, Section 49-3.02A(4)(b).

CIDH Pile Preconstruction Meeting **Agenda / Minutes**	Date: Time: Location:	Project Stamp:

Facilitator:	Structure Representative:
Invitees:	By phone: Resident Engineer: Assistant Structure Rep.: Foundation Testing Branch Rep.: CIDH Pile Mitigation Committee Rep.: Structural Designer: Geotechnical Designer: Contractor's Project Manager: Drilling Subcontractor's Project Manager: Project Superintendent: Drilling Subcontractor's Superintendent/Foreman: Reinforcing Steel Subcontractor's Foreman/Superintendent:
Purpose	Establish contacts and communication protocol for the contractor, the engineer, and their representatives involved in CIDH pile design and construction, and to afford all parties a common understanding of the construction process, acceptance testing, and mitigation process of CIDH piles.

Time	Topic*	Who
	1. Welcome and Self Introduction	
	2. Project Background	
	3. Pile Installation Plan	
	4. Emergency Plan	
	5. Acceptance Testing	
	6. Pile Design Data Form	
	7. Mitigation Process	
	8. Timelines and Critical Path Activities	
	9. Special Structural, Geotechnical, and Corrosion Design Requirements	
	10. Future Meetings	
	11. Safety	
	12. Adjourn Meeting	

* These topics are reminders only. Items will or will not be included depending upon their applicability to a specific project.

Topic 1: Welcome and Self Introduction
a. Attendance Sheet (see attachments) b. Introduction statements about each person's responsibilities during construction of CIDH piles.

Topic 2: Project Background

Topic 3: Pile Installation Plan
Pile Installation Plan Submittal Review: a. Concrete mix design, certified test data, and trial batch reports. b. Drilling or coring methods and equipment. c. Proposed method for casing installation and removal, if necessary. d. Plan view drawing of pile showing reinforcement. Include inspection pipes on the drawing, if inspection pipes are required. e. Methods for placing, positioning, and supporting bar reinforcement and inspection pipes. f. Methods and equipment for determining: f.1 Depth of concrete f.2 Theoretical volume of concrete to be placed, including the effects on volume if casings are withdrawn f.3 Actual volume of concrete placed (see Attachment No. 5) g. Methods and equipment for verifying that the bottom of the drilled hole is clean before placing concrete. How much loose material will be permitted? h. Methods and equipment for preventing upward movement of reinforcement, including the means of detecting and measuring upward movement during concrete placement activities. For concrete placed under slurry, include complete descriptions, details, and supporting calculations in the pile installation plan for: i. Wet vs. Dry (see Attachment No. 2) j. Concrete batching, delivery, and placing systems, including time schedules and capacities. Time schedules must include the time required for each concrete placing activity at each pile. k. Concrete placing rate calculations. If requested, base calculations on the initial pump pressures or static head on the concrete and losses throughout the placing system, including anticipated head of slurry and concrete to be displaced. l. Suppliers' test reports on the physical and chemical properties of the slurry and any proposed slurry chemical additives, including MSDSs. m. Slurry testing equipment and procedures. n. Methods of removal and disposal of excavation, slurry, and contaminated concrete, including removal rates. o. Methods and equipment for slurry agitating, recirculation, and cleaning

Topic 4: Emergency Plan
a. Sidewall sloughing or water inflow during concrete placement. a. Broken tremie, breach of tremie seal, tremie blockage, tremie removal and reinsertion b. Temporary casing removal, breach of casing seal. c. Delay in concrete delivery, loss of concrete workability. d. Rebar cage movement e. Who is authorized to make the decision to abandon concrete placement and remove rebar cage?

Topic 5: Acceptance Testing
a. CIDH Pile Acceptance Test Request Form (see Attachment No. 7) b. The Dept. performs acceptance testing using GGL to test the concrete density of the pile for homogeneity. c. Run dummy probe prior to testing. PVC tubes must be dry prior to GGL testing. d. Blocked tubes and coring. Blockage at the bottom of the pile may not need coring (See BCM 130-10.0) e. GGL Acceptance Test Report by FTB (distribution list – emails) f. Allow 15 days for providing GGL Acceptance Test Report g. CSL/additional testing selected by FTB. GGL report will address additional testing. Contractor may also do their own testing when the state does not elect to do so. h. Allow Dept. 20 additional days to perform and report CSL/additional testing. Allow Dept. 10 days to review CSL/additional testing by Contractor. i. If pile is rejected, suspend concrete placement until revised installation plan is authorized

Topic 6: Pile Design Data Form (PDDF)
a. "Simple Repair" anomaly does not require a PDDF. b. Section 1 of the PDDF to be completed by the FTB (see Attachment No. 8) c. Section 2 of the PDDF to be completed by the Geotechnical Designer. d. Section 3 of the PDDF to be completed by the Structural Designer. e. Section 4 of the PDDF to be competed by the Corrosion Engineer. f. Allow Dept. 30 days to determine if rejected pile requires mitigation (PDDF completion) g. If CSL/additional testing was performed, remember to add 20 or 10 days per Topic 5h or 5i, respectively. h. If anomaly does not require mitigation Contractor may repair the anomaly or pay administrative deduction i. If anomaly requires mitigation follow mitigation process (Topic 7).

Topic 7: Mitigation Process

a. Pile Mitigation Procedure Flowchart (see Attachment No. 9)
b. Contractor proposal/Caltrans review and approval
b. Simple Repair (ADSC Standard CIDH Anomaly Mitigation Plan "A")
c. Basic Repair (ADSC Standard CIDH Anomaly Mitigation Plan "A")
d. Grouting Repair (ADSC Standard CIDH Anomaly Mitigation Plan "B")
e. Structural Bridging Repair
   e.1 PDDF indicating Basic/Grouting Repair not acceptable
   e.2 CIDH Pile Mitigation VA Meeting (see BCM 130-12.0)
   e.3 CIDH Pile Mitigation Plan Review Meeting (see BCM 130-12.0)
f. Supplementation/Replacement
   f.1 Not feasible to repair pile
   f.2 CIDH Pile Non-Standard Mitigation Meeting (see BCM 130-21.0)
   f.3 CIDH Pile Mitigation Plan Review Meeting (see BCM 130-12.0)

Topic 8: Timelines – Critical Path Activities

a. Acceptance Testing and Mitigation Timeline (see Attachment No. 10)

Topic 9: Structural, Geotechnical, & Corrosion Design Requirements

a. Communicate design/performance requirements (i.e. Did geotechnical designer add one pile diameter to the bottom of the pile?)
b. Verify construction methods do not impact performance requirements
c. Structural Construction Considerations (construction joint, splice zones, isolation casing, column to shaft connection detail in Type II shaft, location of inspection tubes, bundling of longitudinal rebar, rebar splicing, concrete cover)
d. Geotechnical Construction Considerations (end bearing, skin friction, permanent casing, rock socket)
e. Corrosion Construction Considerations (Corrosive soil, lowest ground water elevation).

Topic 10: Future Meetings

a. CIDH Pile Non-Standard Mitigation Meeting (see BCM 130-21.0)
b. CIDH Pile Mitigation Plan Review Meeting (see BCM 130-12.0)

Topic 11: Safety

a. Applicable Construction Safety Orders.
b. For CIDH Piles 30 inches or greater in diameter and deeper than 20 ft', Cal-OSHA Mining and Tunneling Safety Orders apply. (see BCM 145-5.0)
c. MSDS for all drilling slurries and chemical additives

Topic 12: Adjourn Meeting

Action Item No. 1	Who:	Due:
Action Item No. 2	Who:	Due:

Bin List

ATTACHMENTS

1. CIDH Pile Preconstruction Meeting Attendance Sheet
2. CIDH Wet vs. Dry
3. CIDH Drilling & Concrete Placement Record
4. Synthetic slurry Test Record
5. Concreting Yield Plot
6. GGL Inspection Tube Verification
7. CIDH Pile Acceptance Test Request Form
8. Pile Design Data Form (PDDF)
9. Pile Mitigation Procedure Flowchart
10. General Mitigation Timeline

Attendance Sheet *CIDH Pile Preconstruction Meeting* *Date:* *Time:* *Location:*		*Project Stamp*	
Name	**Company**	**Phone**	**E-mail**

CIDH Wet vs. Dry
(Inspection Tube Requirements)

Vertical inspection pipes for acceptance testing shall be provided in all CIDH concrete piling 24 inches in diameter and larger, <u>except</u> when the holes are dry or when the holes are dewatered without the use of temporary casing in a manner that controls ground water.

	Entirely Dry	**Dewatered**	**Slurry**
No Temp. Casing	No Tubes	No Tubes	Tubes
Temp. Casing	No Tubes	Tubes	Tubes

Based on FHWA Publication No. FHWA – IF – 99 – 025, OSC concludes that there are situations in which the use of the dry method for constructing CIDH piles when encountering small amounts of water is appropriate. As a guideline the following two conditions must be fulfilled:

1. Less than 12 inches of water accumulates above the base over an hour period when no pumping is allowed.
2. Maximum 3 inches of water in the bottom of the hole at the time of concrete placement and the pile design does not require end bearing. This must be verified through discussions with the designers to gain full understanding of the foundation design prior to start of pile construction.

JOB STAMP

DEPARTMENT OF TRANSPORTATION

CIDH DRILLING & CONCRETE PLACEMENT

OSC-CIDH01 (REV. 10/2011)

Location _____ Sheet No. _____

Ref Elev _____ Date _____

Inspected by _____

Top Elev _____ Actual Tip Elev _____ Pile Length _____

Specified Tip Elev per Plans _____

Elevation (ft)	Drilled Depth (ft)	Time	Drilling Tool	Depth to Slurry (ft)	Drill string #	Remarks

DEPARTMENT OF TRANSPORTATION

SYNTHETIC SLURRY TEST RECORD

OSC-SLR01 (REV. 10/2011)

JOB STAMP

Location _____ Sheet No. _____

Type of Synthetic Slurry _____ Date _____

Slurry Testing Performed by _____

Inspector(s) _____

Notes _____

MIXING TANK PRIOR TO DELIVERY TO THE DRILLED HOLE

	Acceptable				Measured		Check
Density	<=	_____	pcf		_____	pcf	
Viscosity	_____	to	_____	seconds/quart	_____	seconds/quart	
PH	between	_____	to	_____	_____	pH	
Sand Content	<=	_____	%		_____	%	

DURING DRILLING

Mid-Height

	Acceptable				Measured		Check
Density	<=	_____	pcf		_____	pcf	
Viscosity	_____	to	_____	seconds/quart	_____	seconds/quart	
PH	between	_____	to	_____	_____	pH	
Sand Content	<=	_____	%		_____	%	

Bottom of Hole

	Acceptable				Measured		Check
Density	<=	_____	pcf		_____	pcf	
Viscosity	_____	to	_____	seconds/quart	_____	seconds/quart	
PH	between	_____	to	_____	_____	pH	
Sand Content	<=	_____	%		_____	%	

PRIOR TO FINAL CLEANING OF THE BOTTOM OF THE HOLE

Mid-Height

	Acceptable				Measured		Check
Density	<=	_____	pcf		_____	pcf	
Viscosity	_____	to	_____	seconds/quart	_____	seconds/quart	
PH	between	_____	to	_____	_____	pH	
Sand Content	<=	_____	%		_____	%	

Bottom of Hole

	Acceptable				Measured		Check
Density	<=	_____	pcf		_____	pcf	
Viscosity	_____	to	_____	seconds/quart	_____	seconds/quart	
PH	between	_____	to	_____	_____	pH	
Sand Content	<=	_____	%		_____	%	

FINAL CLEANING OF THE BOTTOM OF THE HOLE

Mid-Height

	Acceptable					Measured		Check
Density	<=	_____	pcf			_____	pcf	
Viscosity	_____	to	_____	seconds/quart		_____	seconds/quart	
PH	between	_____	to	_____		_____	pH	
Sand Content	<=	_____	%			_____	%	

Bottom of Hole

	Acceptable					Measured		Check
Density	<=	_____	pcf			_____	pcf	
Viscosity	_____	to	_____	seconds/quart		_____	seconds/quart	
PH	between	_____	to	_____		_____	pH	
Sand Content	<=	_____	%			_____	%	

PRIOR TO CONCRETE PLACEMENT

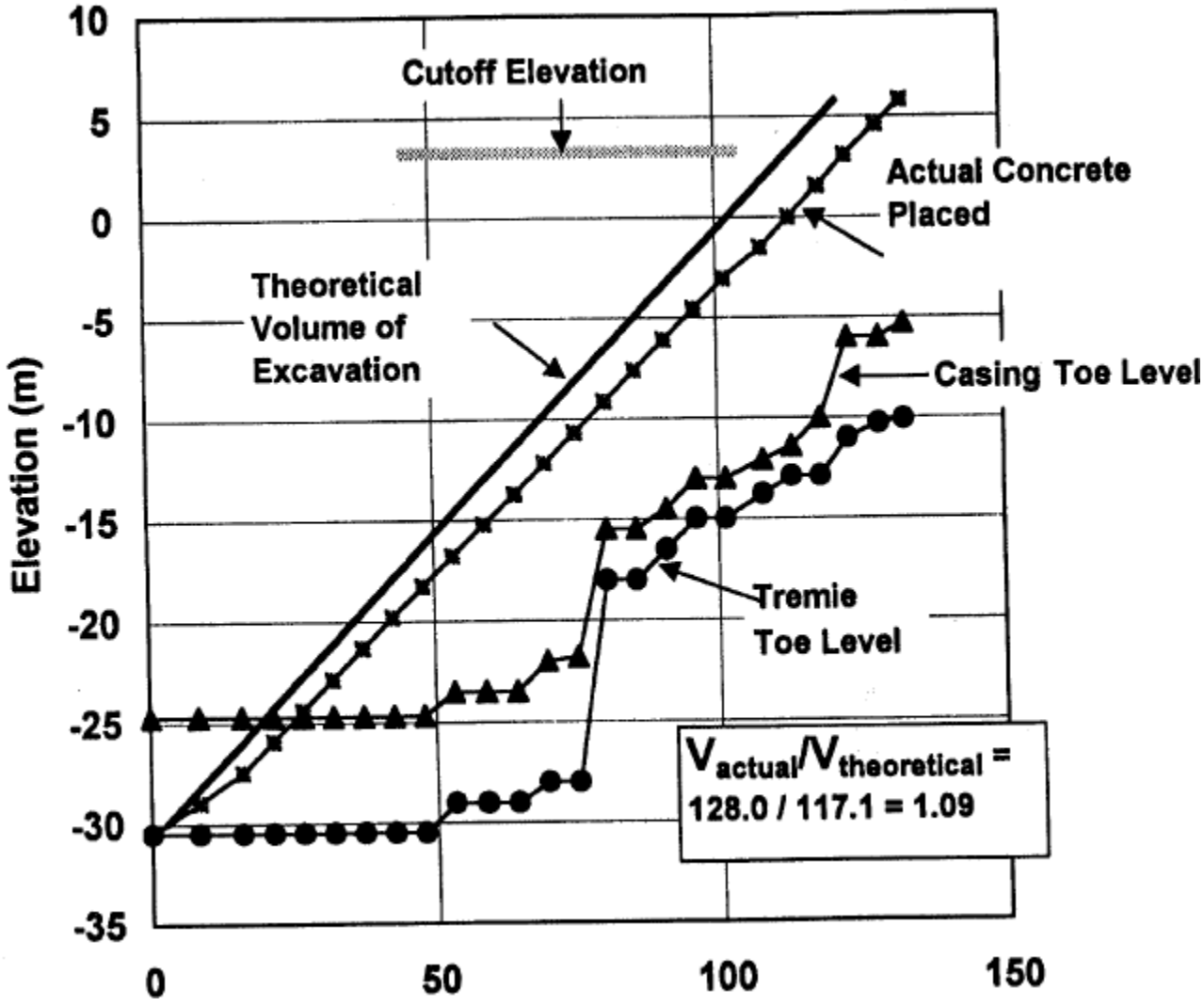
Mid-Height

	Acceptable					Measured		Check
Density	<=	_____	pcf			_____	pcf	
Viscosity	_____	to	_____	seconds/quart		_____	seconds/quart	
PH	between	_____	to	_____		_____	pH	
Sand Content	<=	_____	%			_____	%	

Bottom of Hole

	Acceptable					Measured		Check
Density	<=	_____	pcf			_____	pcf	
Viscosity	_____	to	_____	seconds/quart		_____	seconds/quart	
PH	between	_____	to	_____		_____	pH	
Sand Content	<=	_____	%			_____	%	

Concreting Yield Plot



DEPARTMENT OF TRANSPORTATION

JOB STAMP

GGL INSPECTION TUBE VERIFICATION

OSC-GGL (REV. 09/2010)

LOCATION

INSPECTOR

RIGID CYLINDER TEST PROBE SIZE
(Check box if verified)

☐ 1.25 inch diameter AND 4.5 feet long

☐ Inspection Tubes are Clear

(If inspection tube fails, identify the tube, record the depth of blockage and contact the Foundation Testing Branch)

NOTES

LOCATION

INSPECTOR

RIGID CYLINDER TEST PROBE SIZE
(Check box if verified)

☐ 1.25 inch diameter AND 4.5 feet long

☐ Inspection Tubes are Clear

(If inspection tube fails, identify the tube, record the depth of blockage and contact the Foundation Testing Branch)

NOTES

Fax to: ***Office of Geotechnical Support***
Foundation Testing Branch
Attn: Doug Brittsan

Fax #: (916) 227-1083
Phone #: (916) 227-1079

CIDH Pile Acceptance Test Request Form

Project Name:	Structure Rep:
Dist/Co/Rte/PM:	Office Phone #: ( )
EA No. & Activity Code:	Fax #: ( )
Bridge No:	Pager #: ( )
Date of Request:	Cell Phone #: ( )

Abutment or Bent Location	Pile Numbers	Pile Diameter	Pile Length	Date Ready for Testing	Status
					☐ Estimated Date ☐ Actual Date
					☐ Estimated Date ☐ Actual Date
					☐ Estimated Date ☐ Actual Date
					☐ Estimated Date ☐ Actual Date
					☐ Estimated Date ☐ Actual Date
					☐ Estimated Date ☐ Actual Date

Please submit forms to the above FAX number by 11:00 am Friday for testing to be scheduled during the following week.
Please Update all Estimated dates as dates can be confirmed or as piles become ready for testing.
Inspection tubes must be verified as capable of passing the specified dummy probe prior to a pile being ready for testing.

For FTB Office Use Only

Tracking Number	FT&I Rep	Date Tested	Date of Report	Deadline
Comments:				

For individuals with sensory disabilities, this document can be made available in Braille, large print, audiocassette or computer disk upon request. To obtain one of these alternate formats, please call (916) 227-8185 or TTY 711 or write to the EEO Officer, Division of Engineering Services, P.O. Box 168041, Mail Stop 9 Room 509, Sacramento, CA 95816-8041.

1 Foundation Testing

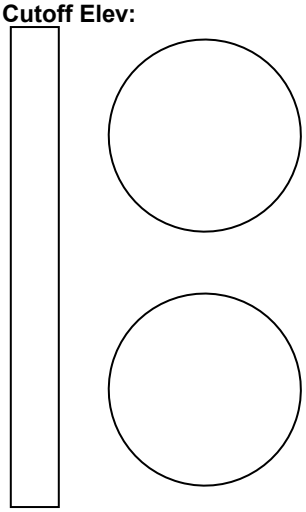
Name:
Phone:
Date:

Anomaly Overview

Testing Performed ☐ GGL ☐ CSL

Shaft Diameter:

Cutoff Elev:



Section A-A

Section B-B

Tip Elev.:

Anomaly Description

Section A-A: _____

_____

_____

Section B-B: _____

_____

_____

2 Geotechnical

Name:
Phone:
Date:

Required Nominal Resistance of Shaft (per contract plans)

Compression:_____kips Tension:_____kips

Lowest Estimated Groundwater Elevation: _____

Remaining Required Nominal Resistance To Be Developed Below Each Anomalous Section:

Section A-A: Compression_____Tension_____kips

Soil and/or Rock Type: _____

Shaft is geotechnically ☐ Acceptable ☐ Unacceptable

Section B-B: Compression_____Tension_____kips

Soil and/or Rock Type: _____

Shaft is geotechnically ☐ Acceptable ☐ Unacceptable

Comments:_____

_____

3 Structural

Name:
Phone:
Date:

As-Designed Capacity of Shaft

Section A-A: Shear: _____ Moment: _____

Section B-B: Shear: _____ Moment: _____

Maximum Demand of Shaft at Section A-A

Shear: _____ Moment: _____

Shaft is structurally ☐ Acceptable ☐ Unacceptable

Maximum Demand of Shaft at Section B-B

Shear: _____ Moment: _____

Shaft is structurally ☐ Acceptable ☐ Unacceptable

Comments:_____

_____

4 Corrosion

Name:
Phone: Date:

Consideration is ☐ Required ☐ Not required

For anomalies between the top of pile and 3 feet below the lowest estimated groundwater level at the site, corrosion results listed in the Geotechnical report are used to assess the need for repair. For situations where results are not available, soil samples may be obtained adjacent to the anomaly and tested in accordance with California Test (CT) 643 (Parts 2, 3 and 4) and if necessary, CT 417 and CT 422 to determine soil corrosivity. For anomalies outside these limits, and where no stray current source can be identified, or for non-corrosive soil conditions, no consideration of corrosion potential is required.

Corrosion Potential at Section A-A: _____

Corrosion Potential at Section B-B: _____

5 Construction

Considering parts 2-4 of this form,

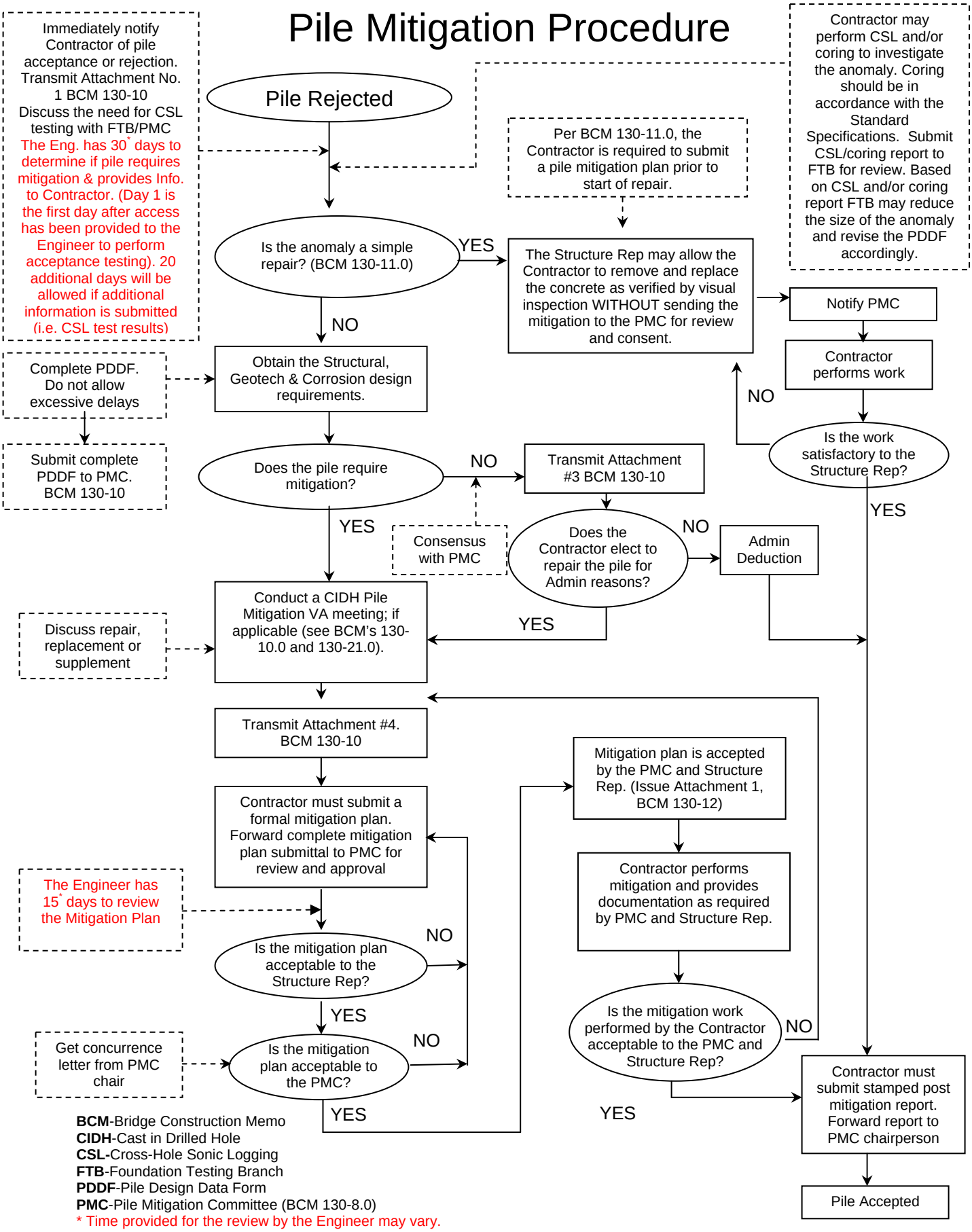
Structure Rep.:
Phone: Date:

Sec. A-A is: ☐ Acceptable with Administrative Deduction ☐ Unacceptable, Mitigation is Required

Sec. B-B is: ☐ Acceptable with Administrative Deduction ☐ Unacceptable, Mitigation is Required

Bridge Name:	Bridge No.:	Abut/Bent:
Dist-Co.-Rte:	EA:	Pile:
Structure Rep.:	Phone:	Fax:

Pile Mitigation Procedure

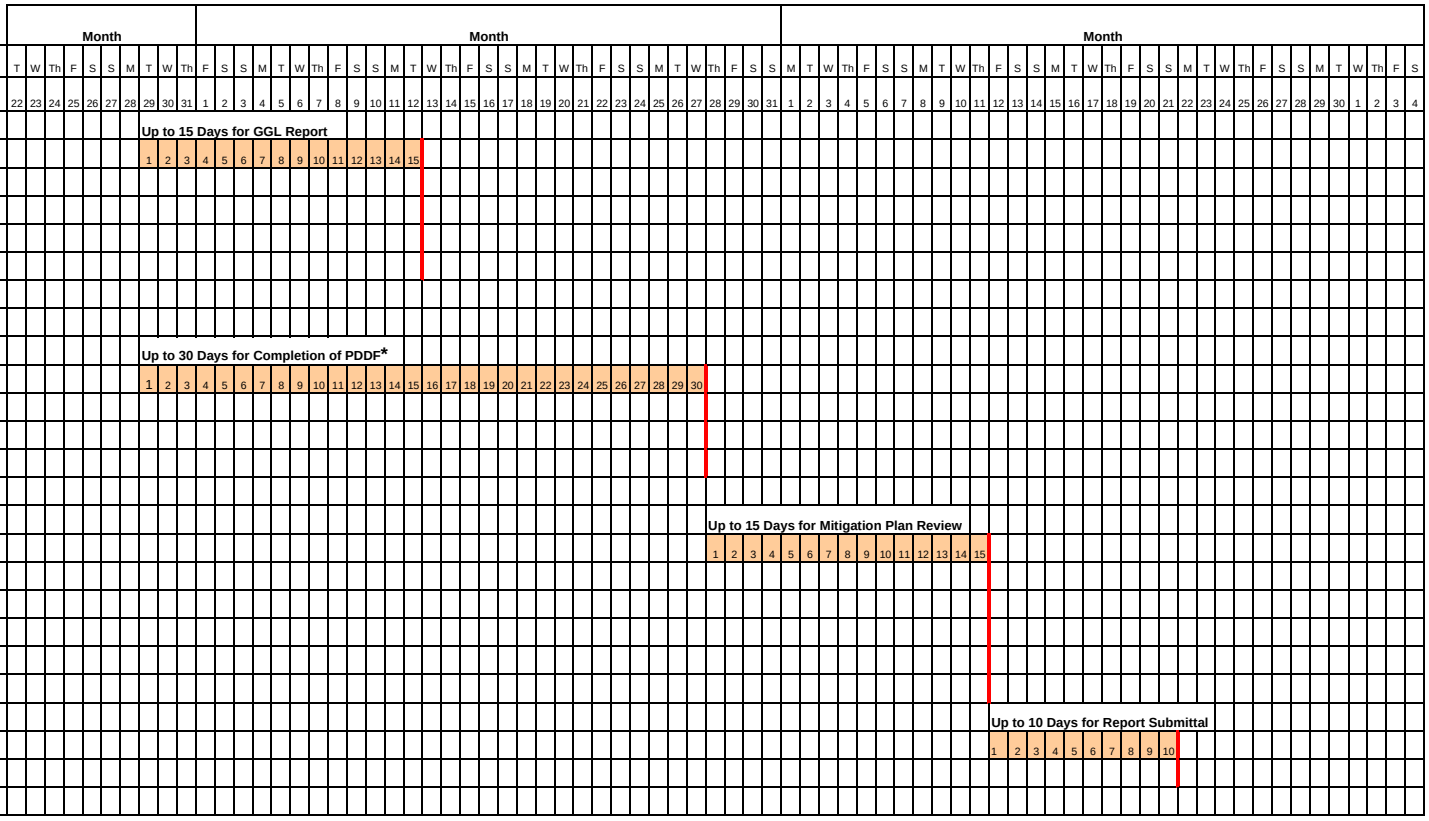


BCM-Bridge Construction Memo
CIDH-Cast in Drilled Hole
CSL-Cross-Hole Sonic Logging
FTB-Foundation Testing Branch
PDDF-Pile Design Data Form
PMC-Pile Mitigation Committee (BCM 130-8.0)

* Time provided for the review by the Engineer may vary.
The Engineer shall review the Standard Specifications for the time allowed.

Mitigation Timeline - General **DRAFT**

DATE		TASK
	1.0	Contractor places concrete
	2.0	Contractor informs SR Pile is ready for GGL
	2.1	SR informs FTB pile is ready for GGL
	2.2	FTB performs GGL testing
	2.3	FTB releases GGL report rejecting Pile
	2.4	GGL report is submitted to Cont.(BCM 130-10 Attach. 1)
	3.0	Is CSL needed? Contractor or FTB performs CSL
	3.1	Contr./FTB releases CSL report, pile remains rejected
	3.2	SR informs Contractor of continued rejection
	4.0	SR forwards PDDF (GGL only, or GGL&CSL) to designers
	4.1	SR forwards revised PDDF (GGL or GGL&CSL) to designers
	4.2	Designers complete PDDF indicating inadequate pile
	4.3	SR informs Contractor of inadequate pile
	5.0	CIDH Pile Non-Standard Mitigation Meeting (see BCM 130-21)
	6.0	SR informs Contr. to submit mitigation plan (BCM 130-10 Attach. 4)
	7.0	Contractor submits complete mitigation plan
	7.1	SR forwards mitigation plan to PMC for review
	7.2	Designers approve mitigation plan
	7.3	FTB approves mitigation plan
	7.4	PMC approval letter to SR
	7.5	SR informs Contr. of plan approval (BCM 130-12 Attach. 1)
	8.0	Pile mitigation operations begin
	9.0	Mitigation completion
	9.1	Contractor submits post mitigation report
	10.0	SR forwards post mitigation report to PMC



* If additional information is submitted to the Engineer that modifies the size, shape, or nature of the anomaly, the Contractor shall allow 20 additional days for the subsequent analysis.

CIDH Pile Non-Standard Mitigation Meeting

The purpose of the *CIDH Pile Non-Standard Mitigation Meeting* is to bring together the Contractor, the Engineer, and their representatives involved in Cast-In-Drilled-Hole (CIDH) pile mitigation to address a N*on-Standard CIDH Pile Mitigation Plan*[1] in a timely manner. It is intended to quickly eliminate any nonviable mitigation methodology and focus all efforts on finding optimal alternatives to mitigate the pile repair. The meeting will provide a forum for free exchange of information so that one or more viable repair solutions can be identified. Identifying these usable repair strategies should not be viewed as directing a Contractor's work or plan. Ultimately, it is the Contractor's responsibility to select, develop, and submit the pile mitigation plan. In many cases, the completion of the CIDH pile is a critical path item affecting the schedule, and in these circumstances it is imperative that the project team communicate effectively so that a satisfactory mitigation plan can be developed and executed with minimal impact on the schedule and delivery of the project. Attendance of this meeting is mandatory for the following:

- Resident Engineer.
- Structure Representative.
- Assistant Structure Representatives.
- Foundation Testing Branch.
- CIDH Pile Mitigation Committee representatives.
- Structural and Geotechnical Designers who are providing construction support for the project.
- Contractor's Project Manager.
- Project Superintendent.
- Drilling Subcontractor's Project Manager.
- Superintendent/Foreman.
- Mitigation Plan Designer.

Attachment 1 is a general meeting agenda to guide you with understanding the key topics that need to be addressed during this meeting for timely development of a non-standard mitigation plan. However, these are reminders only. Review the general meeting agenda with regard to your specific project and modify the meeting agenda as necessary. Certain topics may or may not be included, depending upon their applicability to a specific project.

[1] See BCM 130-12.0, *Mitigation of CIDH Piling*.

CIDH Pile Non-Standard Mitigation Meeting Agenda Minutes

CIDH Pile Non-Standard Mitigation Meeting **Agenda / Minutes**	Date: Time: Location:	Project Stamp:

Facilitator:	Structure Representative:
Invitees:	**By phone:** Resident Engineer: Assistant Structure Rep.: Foundation Testing Branch Rep.: CIDH Pile Mitigation Committee Rep.: Structural Designer: Geotechnical Designer: Contractor's Project Manager: Project Superintendent: Drilling Subcontractor's Project Manager: Drilling Subcontractor's Superintendent/Foreman Mitigation Plan Designer:
Purpose	Bring together the contractor, the engineer, and their representatives involved in CIDH pile mitigation to address a non-standard mitigation plan (replacement, supplementation, or non-standard pile repair) in a timely manner.

Time	Topic*	Who
	1. Welcome and Self Introduction	
	2. Anomaly Description based on GGL, CSL and/or Coring	
	3. PDDF Review (Struc., Geotech. and Corrosion Design Requirements)	
	4. Limitation of Grouting Repair**	
	5. Alternative Repair Methods (i.e. Structural Bridging)	
	6. Supplementation/Replacement	
	7. Discuss successful solution used in past pile mitigation	
	8. Mitigation Plan Design Requirements	
	9. Timelines and Critical Path Activities	
	10. Safety	
	11. Future Meetings	
	12. Adjourn Meeting	

* These topics are reminders only. Items will or will not be included depending upon their applicability to a specific project.

** Any questions regarding a Caltrans decision to not permit grouting may be asked at this time. If a type of repair deemed as non-viable by Caltrans, all further discussions should exclude those options.

Topic 1: Welcome and Self Introduction
a. Attendance Sheet (see attachments) b. Introduction statements about each person's responsibilities during construction of CIDH piles.

Topic 2: Anomaly Description based on GGL, CSL, and/or Coring
a. Characterize the defect – Define Nature, Location, and Extent b.

Topic 3: PDDF Review (Struc., Geotech and Corrosion Design Requirements)
a. Discuss the effect on structural resistance and serviceability. b. Discuss the effect on geotechnical resistance and serviceability. c. Discuss the effect on corrosion design and serviceability.

Topic 4: Limitation of Grouting Repair
a. High pressure water jets are capable of nozzle pressures up to 20,000 psi and can cut limited quantities of concrete at close range if the jet can be directed and is not shadowed by reinforcing steel. It is not normally feasible to remove large quantities of concrete or other semi-structural material in this manner. b. This technique can be used to remediate and improve concrete which has inclusions of soil or low strength concrete. Grouting cannot be expected to restore cross sections in zones of high moment demand. Post-treatment cores or cross-hole sonic logs should show improvement, but will not be free of anomalies. c. Grouting within the shaft may not be effective if the defects to be treated include zones on the outside of the reinforcing cage in granular soils below groundwater. In such a case, attempts to hydroblast outside the shaft would erode unstable soils which might be expected to cave. Jet grouting around the perimeter of the shaft is a technique which might be considered. d. If the shaft is structurally sufficient except for concerns regarding the concrete cover on the reinforcement, or if a void exists between the outside of the shaft and the soil, then grouting around the perimeter may be considered.

Topic 5: Alternative Repair Methods (i.e. Structural Bridging)

a. Increase the structural strength of a defective pile without complete removal of the defect.
b. Install structural steel or pipe section cast into the central portion of the pile with regular or high strength concrete.
c. Additional member designed to restore structural strength to meet design requirements.
d. It may be possible to extend a central drilled section into formation below tip to increase geotechnical capacity of the CIDH pile.
e. Structural enhancement can also be accomplished by drilling holes in the shaft and grouting in additional rebar or high strength bars.
f. Micropiles can be installed by drilling through the pile in order to anchor the pile into underlying formation. It may be possible to install these by drilling through existing inspection tubes.

Topic 6: Supplementation/Replacement
a. In some cases where the strength or stiffness of a drilled shaft is less than required, the most effective remediation strategy might be to add additional deep foundation elements (CIDH, driven, micropile). These might be designed to supplement or even completely replace the defective CIDH pile. b. Incorporating additional deep foundation elements into a common cap with the existing CIDH pile must address the issue of strain compatibility.

Topic 7: Discuss Successful Solution Used in Past Pile Mitigation

Topic 8: Mitigation Plan Design Requirements
a. Provide the Contractor's mitigation plan designer with design information (i.e. moment and shear diagrams) necessary for completion of mitigation plan.

Topic 9: Timelines and Critical Path Activities

Topic 10: Safety
a. Applicable Construction Safety Orders. b. For CIDH Piles 30 inches or greater in diameter and deeper than 20 ft', Cal-OSHA Mining and Tunneling Safety Orders apply (see BCM 14-5.0).

Topic 11: Future Meetings

a. CIDH Pile Mitigation Plan Review Meeting (see BCM 130-12.0).

Topic 12: Adjourn Meeting		
Action Item No. 1	**Who:**	**Due:**
Action Item No. 2	**Who:**	**Due:**

Bin List

ATTACHMENTS

1. CIDH Pile Non-Standard Mitigation Meeting Attendance Sheet

Attendance Sheet ***CIDH Pile Non-Standard Mitigation Meeting*** *Date:* *Time:* *Location:*		***Project Stamp***	
Name	**Company**	**Phone**	**E-mail**

Seal Courses

Seal courses are used to control water so that concrete for footings and pile caps can be placed in the dry condition. Seal courses may also be used at the bottom of open ended Cast-In-Steel-Shell (CISS) piles to maintain water control, especially when high hydraulic heads and permeable soil plugs are present. Occasionally the use of a seal course is included in the design of the footing or pile cap, and is shown on the plans. Seal courses used to facilitate the construction of falsework or structure excavations will not be shown on the plans but can be used by the Contractor in lieu of dewatering methods.

Specifications for seal courses shown on the plans are different than for those done at the Contractor's option. For seal courses, the topics of *Water Control and Foundation Treatment*[1], *Concrete Placed Under Water*[2], and *Payment*[3 & 4], are covered in different sections of the Standard Specifications (SS).

The thickness of the seal course within a cofferdam depends on the hydrostatic head. Seal course thickness calculations can be found in the *Caltrans Foundation Manual, Appendix I.*

When seal course concrete is shown on the plans (with limits and thicknesses), there is typically an item for seal course concrete in the Engineer's Estimate. If not, then it should be addressed in the provisions for *Structural Concrete, Bridge Footing*.

Full compensation for seal course concrete, not shown on the plans for excavations and cofferdams, is considered to be included in the price paid for structure excavation or the contract price paid for the item of work requiring excavation, and no separate payment is made.

Seal course limits and thicknesses need to be shown.

[1] 2010 SS, Section 19-3.03D, *Water Control and Foundation Treatment*, or 2006 SS, Section 19-3.04, *Water Control and Foundation Treatment*.
[2] 2010 SS, Section 51-1.03D(3), *Concrete Placed Under Water*, or 2006 SS, Section 51-1.10, *Concrete Deposited Under Water*.
[3] 2010 SS, Section 19-3.04, *Payment*, or 2006 SS, Sections 19-3.07, *Measurement*, and 19-3.08, *Payment*.
[4] 2010 SS, Section 51-1.04, *Payment*, or 2006 SS, Section 51-1.22, *Measurement*.

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BRIDGE CONSTRUCTION MEMO 132-0.0

MISCELLANEOUS BUILDINGS

June 29, 2010

Volume II

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132-2.0	12/10/1987	JOINT REVIEW OF BUILDING PROJECTS
132-2.1	05/01/1998	INSPECTION OF BUILDING-RELATED TRANSPORATION FACILITIES
132-2.2	07/29/2010	CLEAN RENEWABLE ENERGY BOND (CREB) PROJECT INSPECTION
132-3.0	08/05/1985	PARTS LISTS, SERVICE INSTRUCTIONS, MANUFACTURER'S WARRANTIES, AND OPERATING AND MAINTENANCE INSTRUCTIONS
132-4.0	05/10/1982	CHANGES INVOLVING BUILDING PROJECTS

ROBERT A. STOTT, Deputy Division Chief
of Structure Construction

Offices

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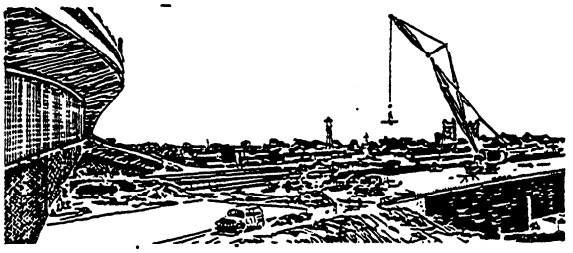


Volume II

WORKING DRAWINGS AND MATERIAL SUBMITTALS FOR BUILDINGS

The submittal of shop plans or equipment lists as specified in the Standard Specifications, the General Conditions, and/or the Special Provisions, requires that each submittal be completely identified. The Structure Representative should caution the Contractor early in the contract (this could be done at the pre-job conference) that this is a contract requirement, and that failure to comply will result in delay of approval of his submittals. (Attachment No. 1 to this Bridge Construction Memo is a sample of an equipment submittal which properly identified the equipment that the Contractor proposes to use.) The Contractor should also be informed that failure to make submittals that are not complete and not grouped in logical order also tends to delay the approval process.

The procedure for review and approval of working drawings and material submittals for buildings is a coordinated effort between the Architecture and Transit Branch, and Construction.

Attached is Memo to Architects A-5-11 (Attachment #2) "Review of Working Drawings and Material Submittal for Buildings". The design memo covers the procedures required for review and approval of working drawings and material submittals, including responsibilities of Structure Representatives on construction projects. Structure Representatives should comply with the applicable instructions in Memo to Architects A-5-1 in-so-far as possible.

Unless otherwise stipulated in the Special Provisions, the Contractor (subcontractor or fabricator) is to submit all working drawings and material submittals directly to the Office of Structure Design, Document Unit, P. 0. Box 942874, Sacramento 94274-0001. This includes original submittals and resubmittals. The Structure Representative is not to accept submittals unless it is so stipulated in the Special Provisions.

The working drawing and material submittals approval procedure is administered by the Special Services Group of Office of Structure Design. The group maintains a record of all working drawings and/or material submittals, and distributes copies to all interested parties, during all phases of the approval procedures. This relieves the Structure Representative of tedious administrative details necessary to insure that working drawings are distributed to the right people at the right time. One copy of all submittals will be forwarded to the Structure Representative on the same day that they are received in Sacramento.

The responsibility for checking working drawings and material submittals is shared by the Architect and the Structure Representative. Working drawings and/or material submittals shall not be returned to the Contractor until the Architect has discussed and resolved the details with the Structure Representative. The comments returned to the Contractor must be acceptable to both the Architect and the Structure Representative.

Agastat timing relays 7000 series

Contractor to show complete contract identification on each copy of each separate submittal. If submittals are not bound permanently in a folder, each sheet is a separate submittal.

BRIDGEPORT MAINT. STATION
09-MND-5722
CONT. NO. 09-056404

Model Number Code

TDR2
(REF. p. 88 SPEC. PROV.)

Contractor to completely identify item.

70 — AGASTAT® 7000 Series timing relay

1 — Operation
- 1 - On-Delay
- 2 - Off-Delay

2 — Contact Arrangement
- 1—Single Pole, Double Throw
- 2—Double Pole, Double Throw
- ① 4—Four Pole, Double Throw

A — Coil Voltage

A	120 V 60 Hz, 110 V 50 Hz
B	240 V 60 Hz, 220 V 50 Hz
C	480 V 60 Hz
D	550 V 60 Hz
E	24 V 60 Hz
F	127 V 50 Hz
G	240 V 50 Hz
H	12 V 60 Hz
I	6 V 60 Hz
J	208 V 60 Hz
K	Dual Voltage (combines A & B)
M	28 VDC
N	48 VDC
O	24 VDC
P	120 VDC
Q	12 VDC
R	60 VDC
S	250 VDC
T	550 VDC
U	16 VDC
V	32 VDC
W	96 VDC
Y	6 VDC
Z	220 VDC

D — ③ Time Range

A	.1 to 1 Sec.
B	.5 to 5 Sec.
C	1.5 to 15 Sec.
D	5 to 50 Sec.
E	20 to 200 Sec.
F	1 to 10 Min.
H	3 to 30 Min.
② I	6 to 60 Min.
② J	3 to 120 Cyc.
K	1 to 300 Sec.

T — Aux. Switch Options
- L
- ① LL
- T

GZ — Optional Features
- A —Quick Connect Terminals
- ①② B —Plug-In Connectors
- ①② C —Plug-In Receptacle (Screw Term.)
- ①② D —Plug-In Receptacle (Quick-connect Term.)
- ① GZ—Total Enc., W/Bottom Connection
- ①② H —Herm. Sealed (Consult Factory)
- ② M —Dustight
- O —CSA Approval
- ① W —Watertight Enc., (NEMA-4)
- ② X —Panelmount Kit

—OPTIONAL—

① Not available on Panelmount models
② Not available on Four Pole models
③ Model 7014 available with letter-calibrated dial only.

Note: As shown above, the Contractor must show all options, accessories, and modifications to be furnished. Arrows, circles, or written notes may be used to identify the characteristics of the furnished item.

BRIDGE CONSTRUCTION MEMO 132-1.0
ATTACHMENT NO. 1 (10-21-81)
SHEET 1 OF 1

Revised Since Last Printing

65F

Review of Shop Drawings
and Material Submittal
for Buildings.

MEMO TO ARCHITECTS:

Procedure

The instructions in this memo apply to shop drawings and material submittals for buildings. Generally, this will apply to all projects prepared by Architectural Design.

Note:

The procedures covered in this Memo will also apply to Mechanical & Electrical (Building or related) projects. The responsibilities may be covered appropriately by inserting "M & E Engineer" whenever any reference is made to "Architect."

Structure Representative = Construction representative or Resident Engineer.

To provide uniform treatment in checking shop drawings and material submitted for buildings, the following procedure shall be followed:

1. The responsibility for checking shop drawings is shared by the Architect and the Structure Representative. Shop drawings shall not be returned to the Contractor until the Architect has discussed and resolved the details with the Structure Representative. The comments returned to the Contractor must be acceptable to both the Architect and the Structure Representative.

   A brief file memo shall be written by the Architect to document controversial decisions or to keep other involved parties informed. For example, a memo is required for any change or clarification of details in contract plans. A copy of the memo is to be sent to the Structure Representative.

2. All submittals of shop drawings and materials will be received by the Documents Unit for distribution. The initial distribution of drawings will be:

   1 copy to RE or Structure Rep
   *4 copies to Architect

   *Including Mechanical/Electrical submittals, Structural submittals, and Landscape/Irrigation submittals.

   Architect will make distributions.

Replaces Memo to
Architects A-5-1 dated
September 1981,

The number of samples of each material may vary. The Documents Unit will submit all samples to the Architect, who will determine and record the disposition.

The Architect will check the drawings and:

(a) Sheets that do not requite correction. Al1 four copies will be stamped "Approved" and distributed as shown below.

PRINTS REVIEWED BY STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
OFFICE OF STRUCTURES – TRANSPORTATION ARCHITECTURE

APPROVED

PURSUANT TO SECTION 5–1.02 OF THE STANDARD SPECIFICATIONS

_____ DATE_____

1 copy will be retained by the Architect
3 copies will be returned to the Documents Unit who will make the following distribution:

1 copy to RE or Structure Rep
2 copies. to Contractor

(b) Sheets that have minor corrections. The Architect will stamp each of these sheets as shown and indicate all corrections in red on all four copies. The distribution will be the same as 2a.

PRINTS REVIEWED BY STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
OFFICE OF STRUCTURES – TRANSPORTATION ARCHITECTURE

APPROVED

SUBJECT TO NOTATIONS INDICATED IN RED

PURSUANT TO SECTION 5–1.02 OF THE STANDARD SPECIFICATIONS

_____ DATE_____

BRIDGE CONSTRUCTION MEMO 132-1.0
Attachment No. 2 (8-5-85)
Sheet 2 of 6

66B

(c) Sheets that have revisions. Only those sheets that require the corrections will be stamped as shown, Notes added to the sheet shall make it clear why the submittal is not approved. The distribution will be the same as 2a.

PRINTS REVIEWED BY STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
OFFICE OF STRUCTURES DESIGN

NOT APPROVED
PLEASE RESUBMIT

DATE _____

(d) Corrected copies received from the Contractor will be processed identical to the procedures outlined in 2, 2(a), 2(b) or 2(c) as required.

3. Far cantracts under General Conditions, the following stamps will be used under 2(a), (b), or (c):

(a)

PRINTS REVIEWED BY STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
OFFICE OF STRUCTURES DESIGN

APPROVED

PURSUANT TO SECTION 2-1.04 OF THE GENERAL CONDITIONS

DATE _____

(b)

PRINTS REVIEWED BY STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
OFFICE OF STRUCTURES DESIGN

APPROVED

SUBJECT TO NOTATIONS INDICATED IN RED
PURSUANT TO SECTION 2-1.04 OF THE GENERAL CONDITIONS

DATE _____

(c)

PRINTS REVIEWED BY STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
OFFICE OF STRUCTURES DESIGN

NOT APPROVED
PLEASE RESUBMIT

DATE _____

4. For contracts under General Specifications (under $25,000.), the following stamps will be used under 2(a). (b), or (c):

(a)

PRINTS REVIEWED BY STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
OFFICE OF STRUCTURES DESIGN

APPROVED
PURSUANT TO SECTION 5.02 OF THE GENERAL SPECIFICATION

DATE _____

(b)

PRINTS REVIEWED BY STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
OFFICE OF STRUCTURES DESIGN

APPROVED
SUBJECT TO NOTATIONS INDICATED IN RED
PURSUANT TO SECTION 5.02 OF THE GENERAL SPECIFICATION

DATE _____

(c)

PRINTS REVIEWED BY STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
OFFICE OF STRUCTURES DESIGN

NOT APPROVED
PLEASE RESUBMIT

DATE _____

Guide for Checking Shop Drawings

As a means of establishing uniform practice and avoiding omissions, but not as a substitute for common sense, the following outline is submitted as a general guide for the checking of the shop drawings. Some items are included which should fall within the duties of the Structure Representative. An overlap may avoid an oversight. These items will be reviewed in the discussion between the Architect and the Structure Representative.

If the individual Project Architect feels that certain factors need not be considered, or that others should be added, it is his prerogative to do so, providing the Structure Representative is agreeable.

1. Read the Standard Specifications (or General Conditions) and the Special Provisions for the particular job. The Special Provisions may modify the usual procedure. Read the correspondence file. There may have been changes approved by the Office of Construction since the contract was let. Call Structure Representative to establish a working relationship and to become familiar with any pending changes or special problems.

2. Changes from the contract plans or specifications, regardless of magnitude, should not be allowed unless they have been discussed and approved by the Structure Representative and the Project Architect. Revisions may be satisfactory structurally or architecturally, but create administrative problems. Changes requiring Contract Change Orders as determined by the Structure Representative need special attention. These change orders could be grouped into two categories:

   (a) Those involving changes requested by the State and minor changes requested by the fabricator where there is no question on approval of the change order by both parties. The shop drawings can be approved, hut the note "Contract Change Order to be processed" must be added to each detail sheet involved,

   (b) Those involving changes requested by the Contractor, other than those covered in 2a. These should be returned to the contractor with the note "Request must be made by the contractor for a Contract Change Order." The Contractor may ask that the shop drawings be held by Architectural Design pending such negotiation. 'Architectural Design should not hold any plans without such a request.

BRIDGE CONSTRUCTION MEMO 132-1.0
Attachment No. 2 (8-5-85)
Sheet 5 of 6

68 F

3. All submittals shall be properly identified. This is covered in the Special provisions under Section 2-l.04 General Condition projects and 12-1.06 of Standard Specification projects. Bridge Construction Memo 132.1.0 instructs the Structure Representative to caution the Contractor of the importance of this identification (this could be done at the pre-job conference). Tabulated below are specific actions to be taken to assure proper identification:

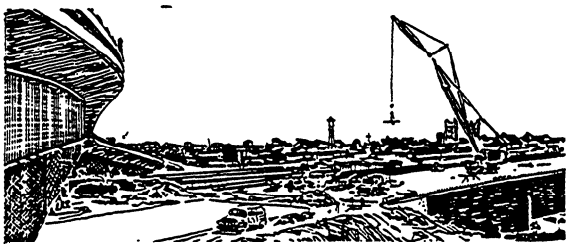
   a. If the project designer does not attend the pre-job conference be sure to notify the R.E. prior to the meeting to stress the importance of properly identifying shop plans.

   b. Inform the Contractor (via the Documents Unit) on the first returned transmittal of sets received with marginal identification. A call to the R.E. to discuss the problem is also in order at this time.

   c. Return seriously unidentified shop plans (via the Documents Unit) without checking (unstamped) when the practice continues after sufficient warning. This option should only be used as a last resort and only after getting the Senior's approval. When you exercise this* option a call to the R.E. and H.0. construction is mandatory.

Earl R. Latham

BRIDGE CONSTRUCTION MEMO 132-1.0
Attachment No. 2 (8-5-85)
Sheet 6 of 6



Volume II

JOINT REVIEW OF BUILDING PROJECTS

Near the completion of a building project, the Structure Representative should arrange for a joint review of the project with representatives of other organizations who have a vested interest in the facility. The purpose of this review is to accomplish the following:

(1) Review the operation of the facility.

(2) Inform the Maintenance Regional Manager or the operators of the facility of the beginning date of the one year guarantee period and who to contact for guarantee work (this should also be covered in the transmittal letter required in Bridge Construction Memo 132-3.0).

(3) Discuss manufacture's warranties, service instructions, etc.

(4) Discuss work that may be required after contract acceptance.

(5) Review all design features that should be handled differently on future projects. These features should also be noted in the comprehensive letter which gives suggestions for improving the design or construction of building projects. (Refer to Bridge Construction Memo 2-8.0.)

The Structure Representative should arrange for the following to attend the review:

(1) Maintenance Regional Manager or his representative for building projects which will be operated and maintained by State Maintenance forces.

(2) A representative of the organization that will be operating and maintaining the facility for building projects not operated and maintained by State Maintenance forces.

(3) The project architect. The Architect will arrange for Headquarters representation at the review in accordance with instruction in the Transportation Architecture Manual (See Attachment No. 1).

At his discretion, the Area Bridge Construction Engineer may determine that minor construction projects do not warrant this joint review. Routine projects having a value under $35,000 such as Minor B contracts would fall into this category. If the review is not held, it is still required that input is obtained from Structure Design and that the appropriate people are informed about the operation of the facility and about the guarantee provisions and who to contact for guarantee work.

It is important that the Maintenance Regional Manager be kept informed regarding job progress on building projects which will be operated and maintained by State Maintenance forces. Therefore, he should be contacted prior to the start of the project work and encouraged to make periodic visits to the job site as the work progresses.

September 1985

A-l-38

Building Project
Joint Review

MEMO TO ARCHITECTS:

The attached Bridge Construction Memo 132-2.0 outlines a joint review upon completion of building projects. The Architect shall arrange for headquarters representation at this review. Typical representation would be:

Off ice of Business Management (OBM) Projects

- Architect
- Structures M & E Engineer(s)
- OBM or H.Q. Maintenance Representation

Safety Roadside Rest Areas

- Architect
- Structures M & E Engineer(s)
- Landscape Architect
- Sanitary Engineer
- H.Q. Maintenance Representation

Truck Weight & Inspection Stations

- Architect
- Structures M & E Engineer(s)
- CHP Representative
- Interagency Liaison with the CHP
- H.Q. Maintenance Representation

Also attached is Bridge Construction Memo 132-3.0 that gives background on parts lists, service instructions, manufacturer's warranties and operating and maintenance instructions.

Earl R. Latham
Design Supervisor

Attachment

BRIDGE CONSTRUCTION MEMO 132-2.0
ATTACHMENT NO. 1 (8-26-85)
SHEET l of 1

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BRIDGE CONSTRUCTION BULLETIN	File: BCM 132-2.1 MISCELLANEOUS BUILDINGS
Approved: _____ R. P. SOMMARIVA, Chief Office of Structure Construction	Date: May 1, 1998 Expires: June 1, 1999 Supersedes: None

Subject: Inspection of Building-Related Transportation Facilities

It is the Structure Representative's responsibility to ensure that all building-related transportation facilities work complies with the plans and specifications. However, some of this work is specialized and complex which the Structure Representative may not be totally familiar.

In order to be assured that the construction of building-related transportation facilities complies with the plans and specifications, the Structure Representative is encouraged to arrange a meeting between themselves and the ESC Project Design Team (Architect, Mechanical, Electrical, etc.) prior to start of construction of the facility. This meeting can be arranged by contacting either the Transportation Architecture Branch (Telephone 916-227-3962, calnet 498-3962) or the lead architect for the project as shown on the General Plan Sheet. This pre-construction meeting between the Office of Structure Construction, Transportation Architecture Branch and Electrical, Mechanical, Water and Wastewater Branch should be used to discuss clarifications to the plans or specifications and should be held prior to the pre-construction meeting with the Contractor.

A similar meeting between the above parties should be arranged at the completion of the project to review administrative and technical issues that may need refinement prior to future use of details, as well as procedures that worked well and should be considered for future projects. This meeting should be held in addition to the meeting required for the "Joint Review of Building Projects" as outlined in Bridge Construction Memo 132-2.0.

Structure Representatives are also referred to Bridge Construction Memo 115-1 .0, Inspection of Electrical, Mechanical, Water and Wastewater Work.

C: BCR&P Manual Holders
Consultant Firms
RETravis, Transportation Architecture Branch
DLScharosch, Electrical, Mechanical, Water and Wastewater Branch
BGauger, Construction Program Manager

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 132-2.2** **MISCELLANEOUS BUILDINGS**
Approved: _____ **ROBERT A. STOTT, Chief** **Offices of Structure Construction**	**Date: July 29, 2010** **Expires:** **Supersedes: Previous 132-2.2**

Subject: Clean Renewable Energy Bond (CREB) Project Inspection

California Department of Transportation (Caltrans) has initiated a program dedicating $20 million to solar energy systems at 70 facilities throughout the state, providing California taxpayers an estimated $52.5 million in avoided energy costs over 25 years.

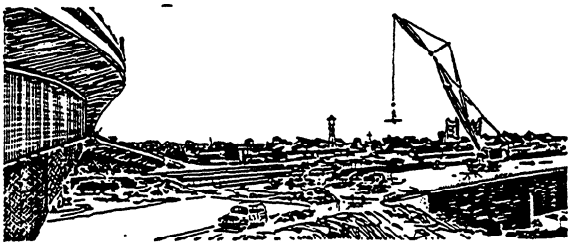
Instead of burning fossil fuels to produce electricity, the photovoltaic panels will harvest energy from the sun, producing more than three million kilowatt-hours of electricity each year and eliminating 2.8 million pounds of greenhouse gases annually.

Construction of these systems will be accomplished by 60 different contracts. The contract types will include Minor A, Minor B, and Major. A few of the contracts will have multiple sites.

It is the Offices of Structure Construction's (OSC) responsibility to provide contract administration and inspection for the CREB contracts. Therefore, the OSC Staff will act as the Resident Engineer (RE) and Structure Representative (SR).

The CREB projects are unique and challenging and a guide has been developed to assist the RE/SR with the technical and administration aspects of the CREB projects. The information in the 'CREB Project Inspection and Administration Guide' will be updated as needed and is located at the intranet link:

http://dschq.dot.ca.gov/OSCHQDownloads/misc/CREB_Admin_Guide.pdf



Volume II

PARTS LISTS, SERVICE INSTRUCTIONS, MANUFACTURER'S WARRANTIES AND OPERATING AND MAINTENANCE INSTRUCTIONS

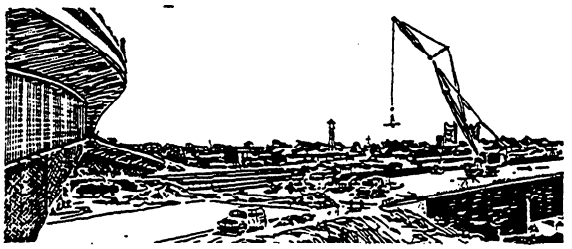
The special provisions for building projects generally require that certain operating and maintenance instructions be submitted in duplicate.

The special provisions for building projects also require that parts lists, service instructions and manufacturer's warranties for products installed in the work shall be delivered to the Engineer.

It is the Structure Representative's responsibility to see that the required parts lists, service instructions, manufacturer's warranties and operating and maintenance instructions are furnished by the Contractor.

If lists, warranties, or instructions are furnished in duplicate, one copy is to be sent to the District Maintenance Engineer and the other copy is to be given to the operators of the facility. If only a single copy is furnished, it should be given to the operators of the facility.

The parts lists, service instructions, manufacturers warranties and operating instructions, furnished to the operator of the facility, and/or to the District Maintenance Engineer, should be accompanied by a transmittal letter. This letter should list all of the instructions, warranties, or parts lists furnished, and give the name, address and phone number of the Prime Contractor. A copy of this letter should be forwarded to the Sacramento Structure Construction Office.



CHANGES INVOLVING BUILDING PROJECTS

Before making any changes involving building projects, the Structure Representative must contact the appropriate section of Structures Design, Architectural and Transit Branch, to inform them of the proposed change, and to obtain their concurrence for the proposed change. Contact with Structures Design may be made either in writing, by telephone, or in person.

When the Contract Change Order Letter of Transmittal is prepared, the Structure Representative must include the following information: Name of person contacted in Structures Design, the method of contacting the person in Structures Design, and a statement that the person contacted concurred with the necessity for, and the provisions of the proposed change.

See Bridge Construction Memo 7-2.0 for additional information concerning Structure Construction change order policy.

BRIDGE CONSTRUCTION MEMO 135-0.0

MISCELLANEOUS CONSTRUCTION MATERIALS

April 11, 2003

Volume II

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135-1.0	01-20-83	EPOXY ADHESIVES
135-2.0	04-16-90	EXPANSION JOINTS AND JOINT SEALS
135-3.0	04-16-90	COMPRESSIVE DEFLECTIONS OF FIBERGLASS OR STEEL REINFORCED ELASTOMERIC BEARING PADS
135-4.0		(BLANK)
135-5.0	4-11-03	MECHANICAL ANCHORAGE DEVICES

DOLORES M. VALLS, Deputy Division Chief
Offices of Structure Construction

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EPOXY ADHESIVES

General Information and Instructions for Use

Epoxy adhesives are very good construction materials. They are, however, somewhat complicated, so there are certain rules that must be followed when using them. Some of these rules are seemingly insignificant, but experience has shown that if they are not closely followed, failure is likely to occur. As a step toward reducing epoxy failures, the adhesive selection, mixing and placing requirements for good performance will be reviewed.

Epoxy adhesives in their pure form are very hard and extremely brittle, and have undesirable properties for structural use. These physical properties can be altered to fit a wide spectrum of hardness and flexibility by judicious selection of hardening and flexibilizing agents. It is only natural, then, that there is a correct epoxy adhesive formulation for each type of job requirement: high strength epoxy to resist high stresses, flexible epoxy to resist high thermal changes.

All State Specification epoxies are of the two component type. One component is the epoxy resin and the other is the hardening agent. To these components are added coloring, stiffening agents, and flexibilizers as required. Some epoxies are designed as 1:l mix ratio of the components, others as a 2:l. They are also designed to have a specific pot life at a certain temperature. When the two components are combined, there are two very important rules which must be followed: (1) mixing proportions as shown on the container must not be changed, (2) the components must be thoroughly blended. One should never try to alter the pot life of an epoxy by changing the prescribed mixing ratio. Doing so would result in a very undesirable epoxy. State Specification epoxies with designed pot lives from a few minutes to about 40 minutes are available. Hence, if the epoxy on hand is either too fast or too slow for a designated job, it should not be used; one which has a pot life that is more compatible with the job requirements should be obtained. The temptation to add solvents to reduce viscosity, extend working time or improve

application characteristics must be avoided. Such usage could cause poor adhesion, high shrinkage and a "cheesy" effect.

The surface receiving the epoxy adhesive must be clean, sound, and dry. Probably the most frequently ignored requirement for a receiving surface is cleanliness. This is especially true when saw-cutting has been done. It is often assumed that a freshly sawed surface is clean since it has been continuously washed by the saw-blade cooling water, when actually a fine residue is left by the water. To insure a clean saw cut surface, it should be sandblasted before application of the epoxy. Cracked or loose sections of concrete near the surface should be located by striking the concrete with a hammer or by dragging. a chain over it. Hollow sounds are indicative of fractured concrete. Loose concrete must be removed.

In respect to strength and permeability, rules which have been developed for portland cement concrete are also applicable to epoxy concrete. The primary difference between the two concretes is that cement, with water for hydration, is the binder in one whereas epoxy is the binder in the other. Similar conditions produce similar results in each. For instance, a uniform gradation of sand and aggregate produces a stronger product than does a single or gap gradation; the richer the mix the less permeable the final product, etc. It follows then that regular concrete sand and gravel, thororoughly dried; usually produces a good epoxy concrete mixture: The maximum size aggregate, as in regular PCC, is determined by job conditions: Depth of section, reinforcing steel restrictions, etc.

The ratio of epoxy to aggregate for a strong, dense mix depends to a great extent on the gradation and maximum size aggregate used. Rule of thumb epoxy/aggregate-ratios (by volume) for good aggregate gradation of various maximum sizes are: l/5 for 1 inch l/4 for pea gravel, and l/3 for 20 mesh sand. Whenever possible, make sample mixes using different ratios of mixed epoxy and aggregates. By comparing these sample mixes, it will be possible to select the mixed epoxy/aggregate mix that is most appropriate to the work. Since the viscosity of epoxies varies considerably with temperature, the mixing ratio of mixed epoxy/ to aggregate may have to be adjusted to maintain workability whenever there is a drop in temperature. When making the sample mixes, use the same compactive effort that it is anticipated will be used when placing the mix on the job. A good mix design is evident if the sample exhibits a mixed epoxy rich surface when

compacted in this manner. A mixed epoxy rich surface results when the mix contains more mixed epoxy than is necessary to coat the aggregate and fill the voids. If an epoxy patch is placed on an exposed deck, an epoxy rich surface will be dangerously slippery.

Required skid resistance can be obtained by broadcasting dry sand on the surface while the epoxy is still fluid enough to receive it.

In an epoxy concrete mixture, it is most important that the epoxy resin components be blended first and then thoroughly mixed into the sand or sand and aggregate. Sides and bottom of mixing containers should be scraped clean during the mixing operation. The mixing operation within the limitation imposed by the pot life must be completed and the mixed epoxy/aggregate placed and compacted before the pot life is exceeded. The results of improper mixing are: non-uniform curing, cheesy or sticky areas, brittle areas poor adhesion and poor performance. The sand and aggregate has to be clean and dry to ensure proper bonding by an epoxy adhesive. Often it is incorrectly assumed that since epoxy will successfully bond wet concrete to dry concrete, good epoxy concrete can be obtained with damp aggregate. This is a false assumption.

In an epoxy rich mixture, there appears to be enough free epoxy to provide bond to the concrete surface against which it is to be placed. However, experience has shown that this is not true. Therefore, to ensure proper bond, the surface must be primed with pure epoxy adhesive just prior to placing the epoxy aggregate concrete.

Epoxy concrete varies as its counterpart PCC in placing characteristics. Epoxy concrete is too sticky and viscous to be effectively vibrated with a spud type vibrator. Surface vibration with a flat plate has not been tried, but may work. The most effective proven way to place epoxy concrete is to work it around reinforcing steel and into corners by hand. Rodding it with a 1 x 1 helps, but generally the springing or bulking characteristic of the material makes rodding somewhat ineffective. Rodding is most effective after the material has been in place a few minutes and some bleeding of the pure epoxy has occurred.

When epoxy concrete is to be placed in thickness greater than 2" it should be placed and compacted in lifts of 2" or less thickness.

The surface of an epoxy concrete can be finished with either a wood float or steel trowel as desired. The steel trowel is superior to a wood float in sealing the surface of an epoxy mixture.

Temperature is a critical parameter in the curing of epoxy, the higher the temperature the faster the cure (and generally the higher the strength.) This fact becomes important when epoxies are used for patches or seals on decks and the controlling factor for opening the deck to traffic is curing of the epoxy.

Curing of epoxy can be accelerated by externally applied heat. Best results for placement in cold weather is to heat the concrete receiving surface preheat the epoxy components and aggregates before any mixing is done and then heat the mixture after it is placed. Preheating the individual components will probably significantly decrease pot life. Experimentally determine the reduction of pot life at the temperature of application. The in-place heating should not be done by direct flame onto the epoxy, but rather by radiant heaters, or by heated air such as is provided by heating a steel plate elevated above the epoxy surface, or by heating the inside of a "tent" erected over the work. Heat lamps directed towards the epoxy is also another good source of heat. The heat of the PCC surface at time of placement, heat of the components before mixing, or heat of the epoxy concrete after placing, should not be greater than about 110° F. When heating the PCC surface, a heater which will not contaminate the surface should be used.

When epoxy is used to bond fresh concrete to hardened concrete, the fresh concrete should be as dry as working condition will allow and must be placed while the epoxy is still fluid. If the epoxy reaches a firm but still tacky state, a new coat of epoxy must be applied onto the hardened concrete before the concrete is placed. If, on the other hand, the epoxy cures beyond the tacky state, it should be sandblasted before the new epoxy coat is applied.

Epoxy concrete dams at expansion joints or epoxy repaired joint spalls should be protected during the epoxy curing period from harmful pressures caused by joint closure as the structure expand. Easily compressed plastic foam materials placed in the joint provides good protection. Forms against which the epoxy concrete is to be placed shall be coated with paraffin or silicone grease, or covered with polyethylene sheet to prevent bond. The epoxy concrete should not be allowed to flow under the forms and

encroach into the space reserved for joint closure. If the material is allowed to flow into this space, failure of the repair is certain when the joint closes against it.

Epoxy injection or pressure sealing cracks with epoxy can be done with 8040-OlE-01, 8040-01E-02, or 8040-0lE-03 material. However, most epoxy injection on contract jobs is done by a subcontractor specalist using epoxy he normally uses for injection work, The Transportation Laboratory should be consulted before permitting the use of other than State Specification epoxies for injection or any other type epoxy work.

The coefficient of thermal expansion for epoxy adhesives is roughly 5 times that of concrete. This large a difference can be tolerated for most epoxies down. to approximately 15° F. because they are still flexible enough at this temperature to "give" under the stress induced by volume change differences. However, since the flexibility of epoxies varies as the temperature, the coefficient difference becomes critical at lower temperatures, Without proper flexibility in the epoxy system, the differential volume change will usually cause shear failure in the concrete, which is generally weaker than the epoxy. There is still sufficient residual flexibility at the lower temperatures in the epoxies designed to bond new concrete to old when their, in-place thickness is l/8 inch or less. These epoxies, however, cannot be used in greater thickness, or in epoxy concrete, or in epoxy mortar; more flexible epoxies are available for these uses.

The exact plastic flow characteristics of epoxy is still undetermined. Hence, until more knowledge is gained on this subject, epoxies should not be used in a manner that will subject them to sustained axial loads.

Where there is no abrasion and a protective coating is required, such as area of edge of deck to drip groove and around scuppers, Design has been using an epoxy enamel such as found in Section 91-4.04 of the Standard Specifications.

Rules, regardless of how complete they may be, are effective only to the extent to which they are followed. In the epoxy use rules discussed, each is a vital link in the process which produces a successful job. Consequently, the degree of success, as measured by in-use performance, of an epoxy application is dependent on the attention given to the adhesive selection, mixing and placing requirements.

For additional information concerning epoxies, contact Tom Shelly (916) 739-2346, ATSS 497-2346, of the Transportation Laboratory.

Safety Precautions

The following precautions are to be followed by all field personnel who are involved with the use of epoxy resin materials:

"The exposed parts of the face, neck and hands should be protected with barrier creams and plastic or rubber gloves be worn during the mixing, blending and placing operations.

"When resins or solvents come in contact with the skin, it should be washed with soap and water, Do not use solvents to clean epoxy from the skin, use soap and water.

"Goggles or face shields should be worn to prevent vapor or liquid splashes from coming in contact with the eyes. If uncured resins or solvents do come in contact with the eyes, they should be flushed continuously for ten minutes and then receive medical attention.

"Contaminated clothing, rags, gloves, etc., should not be reused.

"Good ventilation must be provided for the preparation and use of epoxy resin concrete; and since there entails a fire hazard, fire-fighting equipment must be maintained at all operations."

Following are listed seventeen safety rules which should be followed to offset the hazards inherent in the use of epoxy resins:

1. Inform workers of the hazards of epoxy resin operations and show them how to avoid contact.

2. Provide special isolated areas in the plant for mixing, molding, curing, casting, and tooling of epoxy resins.

3. Install ventilated hoods in mixing areas to prevent the spread of hazardous vapors.

4. Limit mixing to only a few workers.

5. Ventilate grinding, sawing, drilling or polishing operations where epoxy resins are used.

6. Supply protective sleeves and cotton liners under rubber gloves to workers in molding operations.

7. Provide water-soluble skin protective gels.

8. Supply neutral or acid soap instead of alkaline, powdered or abrasive cleansing agents.

9. Don't permit workers to use acetone or solvents to cleanse the skin.

10. Replace clean-up rags with disposable paper towels.

11. Institute a strong housekeeping program that immediately washes up spills and keeps tables, machinery, tools, floors, walls and windows free of particles and dust.

12. Provide goggles and respirators when epoxy resins are sprayed.

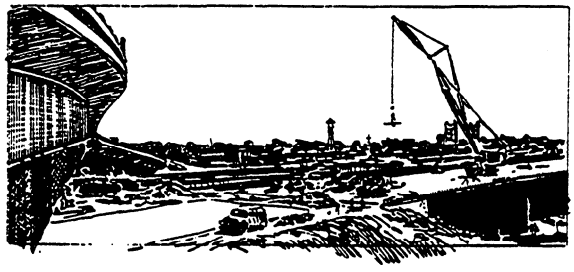
13. Cover benches and seats in mixing areas with disposable paper.

14. Throw away empty epoxy resin containers and drums.

15. Enforce a program of individual worker sanitation which requires washing before eating, before relief periods, after work, and after any contact with epoxy resins.

16. When possible, mechanize blending, mixing, and pouring operations.

17. Prohibit the wearing of clothing soiled by epoxy resins.



BRIDGE CONSTRUCTION MEMO 135-2.0

MISCELLANEOUS CONSTRUCTION MATERIALS

April 16, 1990

Volume II

BRIDGE DECK EXPANSION JOINTS AND JOINT SEALS

CONSTRUCTION POLICY:

a. General

All deck expansion joints and joint seals, except for special cases, will be specified by seal type and M.R. (Movement Rating). The success or failure of joint seals will depend greatly on the enforcement of the specifications. Questions concerning joint seals will be handled in normal channels through the Construction Engineers and the Structure Construction Office.

It is the Structure Representative's responsibility to:

(1) Determine the proper groove width or installation width for the joint seal used, and to complete the applicable portions of the "Joint Movements Calculations" sheet (Form DS-D129).

(2) Install movement recording scribes on all expansion joints.

b. Special Details

Check details such as water stop, formed joint openings, hinge restrainers, rollers or rockers, conduits, etc., for proper setting and movement capacity. All components in an expansion joint must be capable of withstanding more than the anticipated movement for a particular joint.

Joints to be sealed under rehabilitation contracts must first be cleaned of all existing seal material, joint filler, dirt and debris to the top of the waterstop. If the joints do not have a waterstop, or the waterstop is damaged, it is essential that the joint be cleaned down to the bearing or hinge seat. Care should be taken so that existing utilities and encroachments spanning joints are not damaged by the cleaning operations. Carefully inspect the condition of the existing joint and the face of the saw cut. It may not be necessary to resaw cut the joint. If not, a change order may be written to eliminate the saw cutting with a credit to the State.

All dimensions of the existing joint must be verified to be compatible with the new seal, including the depth. All joint damage shall be repaired as directed by the Engineer. Sawcutting or grinding may be required in addition to abrasive blast cleaning of joints. Cleaning joints below the existing damaged waterstop and repairing the existing joint damage shall be considered to be specified extra work. Cost of repair of damage caused by the contractors operations shall be borne by the contractor. Getting a satisfactory joint may require the repairs of spalls, cracks, and expansions dams, and this is usually classed as other work. Supplemental funds should have been provided for all above noted extra work.

c. Saw Cutting

1. Type "A" and "AL" Seals

Joints to be sealed with type "A" Seals are to be saw cut to the dimensions shown on the contract plans. If for some reason the saw cut width has to be increased slightly to maintain a uniform groove width or to expose good sound concrete, it is essential to maintain a 1 to 3 depth to width ratio of the polyurethane seal.

Joints sealed under rehabilitation contracts with type "A" (modified) seals, shall have a groove width $\geq$ one inch and $\leq$ 1.75 inches. Joint seal depth shall equal l/3 the joint width but must be $\geq$ l/2 inch, (see Attachment No. 9).

The 1 to 3 depth to width ratio does not apply to the type "AL" seal. (Saw cut not required).

2. Type "B" Seals

In new construction, type "B" seals are to be saw cut as follows:

Joint movement calculation sheets, which include saw cut information, will be furnished by Design upon the request of the Structure Representative, when they are not included in the R.E. Pending File. (Attachment #2 is an example of a completed Joint Movement Calculation Sheet.)

Saw cutting shall not be started until the Type "B" seal material has been tested and released. The Transportation Laboratory will furnish each job with a copy of the test report showing the M.R. (Movement Rating) of the Type "B" seal groove width limits, (W_1 & W_2) which are necessary to determine the saw cut widths. The M.R. of the Type "B" seal must be equal to or greater than that shown on the contract plans.

The minimum saw cut (groove) depth is to be checked by cutting, a l/2" to 1" section of the actual seal to be used and placing it between two flat surfaces, such as 1"x4"x8", e.g. Place the top of the seal to the dimensions shown on the contract standard plan and compress it to the W_2 position. At this position determine the saw cut depth required per the standard plans.

At the time saw cutting is to begin, determine the groove or saw cut width as described on the joint movement calculation sheet shown in the example (Attachment No. 2). Mark and <u>check</u> the initial saw cut so that it can be used later to check the tolerance of the completed joint. This is very important because the joints are usually moving while the saw cutting is in operation. It is the Contractor's responsibility to adjust the cut accordingly to match the initial saw cut width and maintain the tolerances specified for the completed joint.

In new construction projects joint geometry is readily controllable, i.e. the size of the saw cut is set to accommodate the joint seal. Rehabilitation projects differ from new construction projects in that the width and condition of the joints require special consideration. The new joint seal must provide the required movement rating and also must be of sufficient size to fit the existing joint after saw cutting.

Rehabilitation projects require that both the Minimum W_1 (the maximum joint width at minimum temperature, after prestress shortening), and the M.R. be indicated on the plans. To ensure a correct fit, the W_1 of the joint seal must be greater than the minimum W_1 of the joint.

The Special Provisions require that the joint size be verified prior to ordering the seals. A joint should be remeasured only after that joint and its adjacent joints have been cleaned. Record the concrete temperature at the time of measurement.

Calculate the minimum W, required for the joints using the actual measurements.

$$\text{Min } W_1 = W_e + l/2 + \frac{(T_{str} - T_{min})}{1} \quad (2) \left(\frac{4}{(100)}\right)$$

Where:

Min W_1 = Maximum joint width in inches
W_e = Existing joint width in inches (measured at the widest point)

l/2 = Minimum practical concrete removal (l/4 inch each side of the joint)

T_{str} = Structure temperature, deg F (measured at the time the existing joint width was measured, W_e)

T_{min} = Minimum temperature at structure site - from form DS-D129

1 = Temperature range at structure site - from form DS-D129

2 = Thermal movement in inches/100 feet - from form DS-D129

4 = Contributary length in feet - from form DS-D129

Compare these. recalculated W_1's with the minimum W_1's shown on the plans. If they agree within 0.1 inch, the data shown on the plans does not need to be revised. If the new W_1's do not agree with the values shown on the plans, prepare a contract change order to revise the W_1's and state whether or not the movement ratings have changed.

If a calculated W_1 exceeds 4.25 inches, a compression seal should not be used. Contact the chairman of the Joint Seal Committee for a recommended course of action to follow.

Again, saw cutting should not start until test data for the seal to be used is available. Saw cut widths should be set to provide the minimum joint width possible. Due to the variables involved, saw cut widths should be calculated using the formulas given below and the narrower width chosen, provided it will work.

$$s_1 = W_1 -- \frac{(T_{Str} - T_{min})}{1} (2) \left(\frac{4}{100}\right)$$

$$s_2 = W_2 + \frac{(T_{max} - T_{str})}{1} (2) \left(\frac{4}{100}\right)$$

$$S_3 = w_e + l/2 = \text{Minimum practical saw cut width}$$

Where:

S_1, S_2, S_3, = possible saw cut widths

W_1 = W_1 taken from test report (R-29)

W_2 = W_2 taken from test report (R-29)

W_e = Existing joint width in inches (measured at widest point)

l/2 = minimum practical concrete removal (l/4 inch each side of joint)

T_{str} = Structure temperature, deg F (taken at the time of measurement of W_e)

T_{min} = Minimum temperature at structure site - from form DS-D129
1 = Temperature range at structure site - from form DS-D129
2 = Thermal movement in inches/100 feet - from form DS-D129
4 = Contributary length in feet - from form DS-D129.

d. Installation

1. Tvpe "A" and "AL" Seals:

Be thoroughly familiar with the contract specifications and details and enforce them.

It is essential that the polyethylene foam be placed at a uniform depth to preclude excessively thin or thick sections. There is a successful relationship between the cohesion and the adhesion of the polyurethane seal if the proper shape and dimensions shown on the Standard Plan are maintained. Cut templates out of plywood to check the surface depths of the polyethylene foam and the polyurethane.

Type A (modified) seals require placing the joint seal and rod stock 3 inches up into the curb or rail on the low side of the deck at the curb or rail joint that lines up with the deck joint.

2. Type "B" Seals: (Attachment #l gives the properties for some brands of Type "B" seals.)

Again be thoroughly familiar with the contract plans and specifications and enforce them.

Repair all spalls and grind chamfer in advance of installing the seal.

As a final check, prior to installation, it is recommended to use a thin section of joint seal material and use it to check the saw cut depth throughout the length of joint. Place the seal section in the planned position and check to see that the dimensions shown on the Standard Plan are maintained. Most joint seal failures result from improper saw cuts or from the seal being placed too near the deck surface.
Bend type "B" seals 6 inches up into the curb or barrier rail on the low side of the deck. If the curb or rail joints don't line up with the deck joint, an attempt must be made to abut the joint seal to the face of the curb or rail so that it will provide a water tight seal.

3. Joint Seal Assemblies:

Details of a joint seal assembly are shown on the contract plans. The Structure Representative is to calculate the installation width of the joint seal assembly. Calculations are to be shown on the "Joint Movements Calculations" (DS-D129) sheet using a W_2 equal to l/2 inch minimum at maximum temperature.

The Special Provisions permit alternate joint seal assemblies which the Contractor may use in lieu of the joint seal assembly detailed on the Contract Plans.

If the Contractor proposes to use an alternate joint seal assembly, the Structure Representative shall send two copies of the initially submitted working drawings to Structures Design for a determination as to the adequacy of the proposed alternate joint seal assembly. When submitting the working drawings, point out that they detail a contractor proposed alternate joint seal assembly, and that they are submitted for an informal review by the Joint Seal Committee and by Structures Design.

If an alternate joint seal assembly is incorporated in the contract work, the Structure Representative should make the necessary changes on the "As Built" plans to indicate the details of the alternate joint seal assembly. An additional sheet may be necessary to show the "As Built" details. Do not submit the shop plans as "As Built" plans.

Note that prestressed concrete structures are expected to initially shorten about 0.50 in./100 ft. due to stressing. The total long-term shortening is anticipated to be 1.00 in./lOO ft. for post-tensioned bridges and somewhat less for pretensioned bridges. The difference between the long-term shortening (1.00 in.) and the initial shortening is equal to 0.5 in./100 ft. This is the value shown on the "Joint Movements Ca1culations" form (DS-D129) as "Anticipated Shortening for Post Tensioned Concrete Structures". For unusual situations when a substantial amount of time has elapsed between stressing and the placement of joint seals, an estimate may be made of the amount of prestress shortening that has occurred. Refer to Attachment No. 10 for an example,

4. Modular Joint Seal Assemblies (MR over 4")

Refer to the Special Provisions for details concerning the installation of modular joint seal assemblies. Any questions can be directed to your area Senior or the Joint Seal Committee.

5. Open Joint and Experimental Test Seals

Obtain the necessary brochures on installation procedures from your Construction Senior or the Chairman of the Joint Seal Committee if they are not included in the R.E. Pending File.

The proper installation width of open joints or experimental joint seals will be calculated from the Joint Movement Calculation Sheet. Determine the minimum width at maximum temperature (W_2) and insert this in Column 5. The adjustment of the width for temperature at time of installation will be the same as for the Type B Seal.

e. Expansion Joint Scribes

Scribes are to be placed at all expansion joints as shown on the attached instruction sheet (Attachment No. 3). Placement of the scribes at a location other than that shown may be required when special barrier rails are used. Use the 8" steel railing scribe, 3/4"x8" 24 gauge (Item No. 6635 1760 5) and 4" aluminum scribe plate, 1 1/2"x4" 16 gauge (Item No. 6635 1790 8) for joints having a movement rating of 2" or less. Use the 10" steel railing scribe, 3/4"x10" 24 gauge (Item No. 6635 1780 7) and 6" aluminum scribe plate, 1 1/2"x6" 16 gauge (Item No. 6635 1770 6) for joints having a movement rating greater than 2" Use 681-80-44 Rapid Set Epoxy (Item No. 8040 0100 4) to attach the scribes and plates to the rail. Scribes, plates and epoxy should be obtained from the District through the Resident Engineer. Order one scribe per expansion joint and epoxy at the rate of 1 unit (l/4 pint can of "A" and l/4 pint can of "B") per 20 scribe units. Skewed, or extra wide structures may require a scribe unit on the joint on both sides of the structure.

APPROXIMATE PROPERTIES
FOR
PREFORMED ELASTOMERIC JOINT SEALS
TYPE B_1

Manufacturer's Nominal Properties for Design Data Only
(See Note 4)

Catalog Number (See Note 1)		Uncompressed Size W_0 D_0 (See Note 3)		Approx. M.R. (See Note 2)	Max. Groove Width W_1	Min. Groove Width W_2	Recommended Saw Cut Depth
Brown	H-2503	2.5"	2.625"	1"	2.13"	1.13"	4.0"
Brown	H-3000	3.0"	3.25"	1"	2.55"	1.55"	5.0"
Brown	H-3500	3.5"	3.75"	1.5"	2.98"	1.48"	5.85"
Brown	H-4000	4.0″	4.25"	1.5"	3.40"	1.90"	6.0"
Brown	H-5000	5.0"	5.00"	2"	4.25"	2.25"	7.75"
Brown	H-6000	6.0"	5.5"	2.5"	5.10"	2.60"	9.25"
W.B.	WA-250	2.5"	2.75"	1"	2.13"	1.13"	3.56"
W.B.	WA-300	3.0"	3.38"	1"	2.55"	1.55"	4.31"
W.B.	WA-350	3.5"	3.5"	1"	2.98"	1.98"	4.44"
W.B.	WA-400	4.0"	4.38"	1.5"	3.40"	1.90"	5.00"
W.B.	WA-500	5.0"	5.0"	2"	4.25"	2.25"	5.94"
W.B.	WA-600	6.0"	6.0"	2.5"	5.10"	2.60"	7.75"

*W.B. - Watson Bowman

Notes:

(1) Brand Names other than those listed may be available.

(2) The actual Movement Rating equals (W_1-W_2). W_1 shall be the smaller of the values determined as follows:

1. 0.85 times the manufacturer's designated minimum uncompressed width of the seal (W_0).

2. The width of seal on the third successive test cycle of the pressure-deflection test, when compressed to an average pressure of 3.0 pounds per square inch.

W_2 shall be the width of seal determined on the third successive test cycle of the pressure-deflection test, when compressed to an average pressure of 4 times the pressure measured at the seal width W_1.

(3) Data shown may change significantly due to variations in extrusions. Dimensions must be verified in the field.

(4) Do not use these properties in lieu of actual test results. This is for additional information only. Actual values for W_1, W_2, and M.R. are obtained from test results performed by the Transportation Laboratory on the Report of Inspection of Material (Form TL-29).

DEPARTMENT OF TRANSPORTATION

JOINT MOVEMENTS CALCULATIONS

DS-D129

ROUTE :

1 DESIGNER F. C. Boyd
2 SPECIFICATIONS A. J. Pugh
3 JOINT SEAL COMMITTEE CHAIRMAN C. W. Jones
4 R.E. PENDING FILE S. Matney

AFTER AWARD OF CONTRACT

5 R.E. OR BR: COMPLETE & RETURN TO DIVISION OF STRUCTURES WITH JT SEAL REPORT.
6 JOINT SEAL COMMITTEE CHAIRMAN
7 MAINTENANCE

EXPENSE AUTHORIZATION	DIST	COUNTY	ROUTE	PM	BRIDGE NAME & NO.
03-000000	03	Sac	5	0.2/2.4	Dry Creek O.C. 29-000

TYPE STRUCTURE	TYPE ABUTMENT	TYPE EXPANSION (2" ELASTO PADS ETC.)
Reinf. Conc. Box & CIPP/S	A1 - 70T Piles / A7 - Spd. Ftg.	Bent 3 - Steel Hangers Bent 5 - 2½" Pads

TEMPERATURE EXTREMES (from preliminary report) MAX 110 °F

- MIN 23 °F

TYPE OF STRUCTURE = RANGE 87 °F ①

		② THERMAL MOVEMENT (in./100 ft)	+	ANTICIPATED SHORTENING (in./100 ft)	=	③ MOVEMENT FACTOR (in./100 ft)
STEEL	" (°F) (.0000065)(1200)	=	+	0.00	=	
CONCRETE (Conventional)	" (87 °F) (.0000053)(1200)	= 0.55	+	0.06	=	0.61
CONCRETE (Pretensioned)	" (°F) (.0000053)(1200)	=	+	0.12	=	
CONCRETE (Post tensioned)	" (87 °F) (.0000053)(1200)	= 0.55	+	0.50	=	1.05

ⓐ TO BE FILLED IN BY DESIGNER: F. C. Boyd DATE 9-1-74

TO BE FILLED IN BY BR: W. T. Trustworthy DATE 7-4-76 & 7-5-76

LOCATION	SKEW DO NOT USE IN CALC	④ CONTRIB LENGTH (Ft)	CALCULATED MOVEMENT (Inches) ③ × ④/100	M.R. (Inches) ROUND UP TO ½"	SEAL TYPE A, B, (OTHERS) OR OPEN JOINT	ⓑ SEAL WIDTH LIMITS: CATALOG NUMBER	W1 (Inches) MAX.	⑤ W2 (Inches) MIN. MAX TEMP	ⓒ GROOVE (SAW CUT) WIDTH OR INSTALLATION WIDTH: STRUCTURE TEMP. (°F) *	⑥ ADJUST FROM MAX TEMP (Inches) Δ°/① × ② × ④/100	INSTALLATION OR GROOVE WIDTH AT TEMP. LISTED (Inches) W = ⑤ + ⑥
Abut. 1 (Conv.)	0	64	0.39	½	A	PRC 3105					1.50
Span 3 Hinge (Conv.)	0	166	1.01								
Span 3 Hinge (CIPP/S)	0	220	2.31								
Span 3 Hinge Total	0	386	3.32	3½	Joint Seal Assembly	Arm. Neoprene	4.50	0.50**	90	0.49	0.99
Span 5 Hinge (CIPP/S)	0	100	1.05								
Span 5 Hinge (Conv.)	0	156	0.95								
Span 5 Hinge Total	0	256	2.00	2	Type B	Brown H-5000	4.25	2.00	94	0.26	2.26
Abut. 7 (Conv.)	0	34	0.21	½	A	PRC 3105					1.50

ⓐ SHOW LINE DRAWING OF STRUCTURE ON REVERSE SIDE; SHOW POINTS OF NO MOVEMENT AND CONTRIBUTORY LENGTHS. RETAIN COPY FOR DESIGN CALCULATIONS FILE.

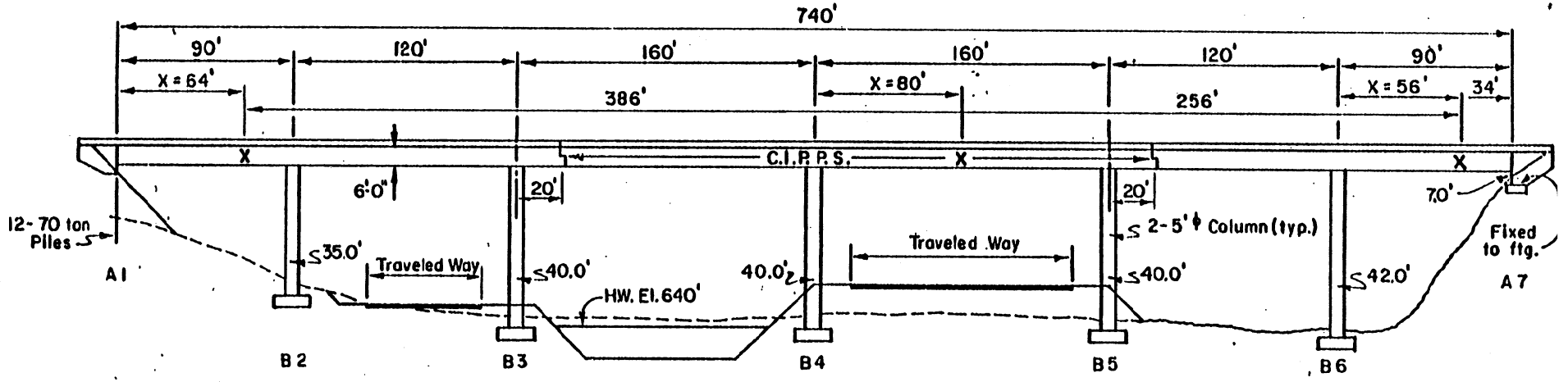
ⓑ INFORMATION FROM TRANSPORTATION LAB REPORTS ⓒ GROOVE WIDTH ADJUSTMENT BASED ON Δ° = (MAX TEMP EXTREME) MINUS (SUPERSTRUCTURE TEMPERATURE).

* MEASURE SUPERSTRUCTURE TEMPERATURE BY PLACING BULB OF CONCRETE THERMOMETER 6" ± INTO EXPANSION JOINT.

** If alternative assembly is used, select correct W2 from sketches shown on Attachment No. 5

BRIDGE CONSTRUCTION MEMO 135-2.0 ATTACHMENT NO. 2 (Rev.9-15-78) Sheet 1 of 2

CALCULATION OF POINTS OF NO MOVEMENT



I (Ft)^4	1.38	61.36	61.36		61.36	61.36		61.36		102	
L (Ft)	5.50	35.0	40.0	Σ	40.0	40.0	Σ	42.0		7.0	Σ
P (kips) @ 1" side sway	1200	618	415	2,233	415	415	830	359	Will slide	600	959 ≈130,000
D (dist. from 1 st. member of frame)	0	90	210		0	160		0		90	
P×D/100	0	556	870	1,426	0	664	664	0		540	540
$X = \frac{\Sigma P \times D/100}{\Sigma P}(100') = \frac{1426}{2233}(100) = \underline{64'}$					$\frac{664}{830}(100) = 80'$			$\frac{540}{959}(100) = \underline{56'}$			

Notes:

Width Str. = 78'
Dia. Col. = 5'-0"
K / Pile @ 1" defl. = 100
X = Point of No Movement

Assumptions:

1. Super str. inf. rigid
2. Col's fixed top & bottom
3. Abut. diaph. will slide @ a force equal to D.W.
4. E (piles) = 4×10^6
   E (columns) = 3×10^6

P (Col.) = $12\ EI \frac{\Delta}{L^3}$

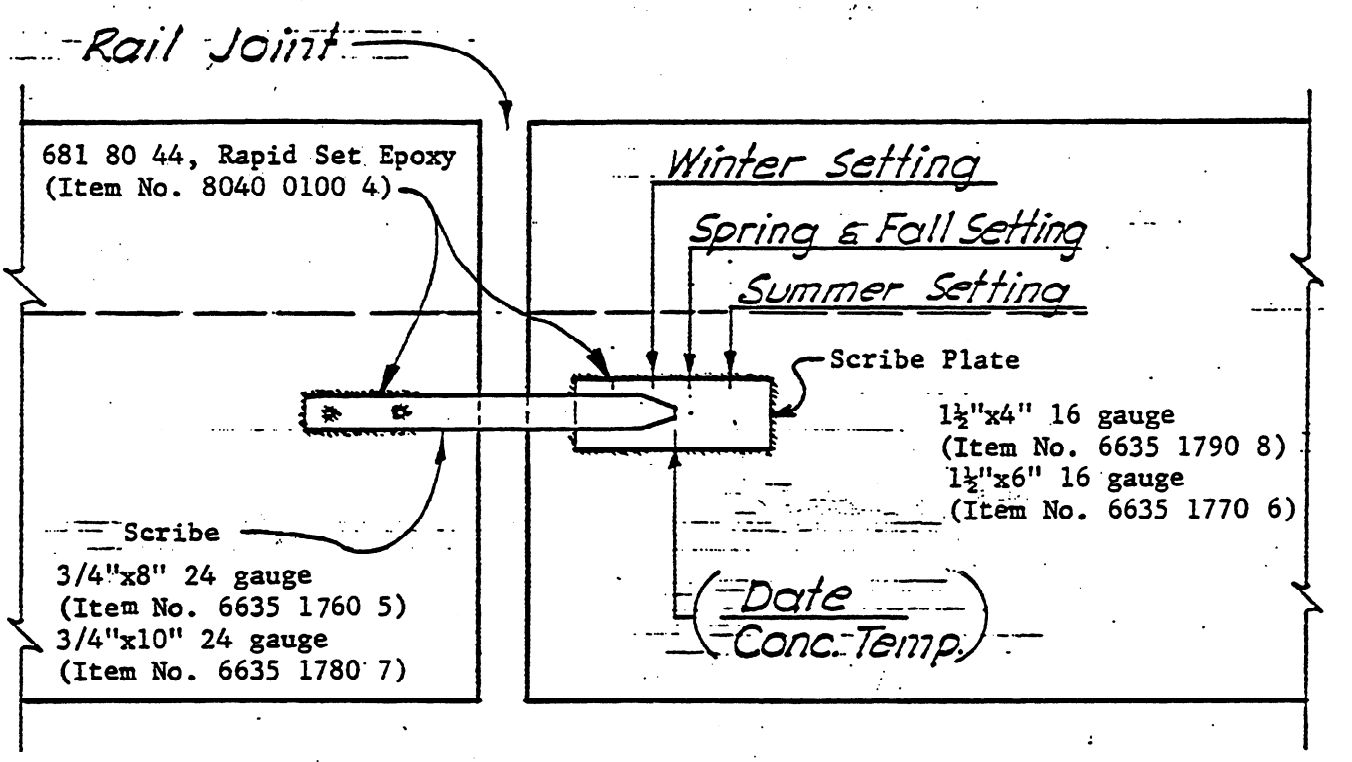
@ 1" defl. = $\frac{432\ I}{(\frac{L}{10})^3}$

D.W. Abut 5 = 600K (assume linear up to 1" defl.)

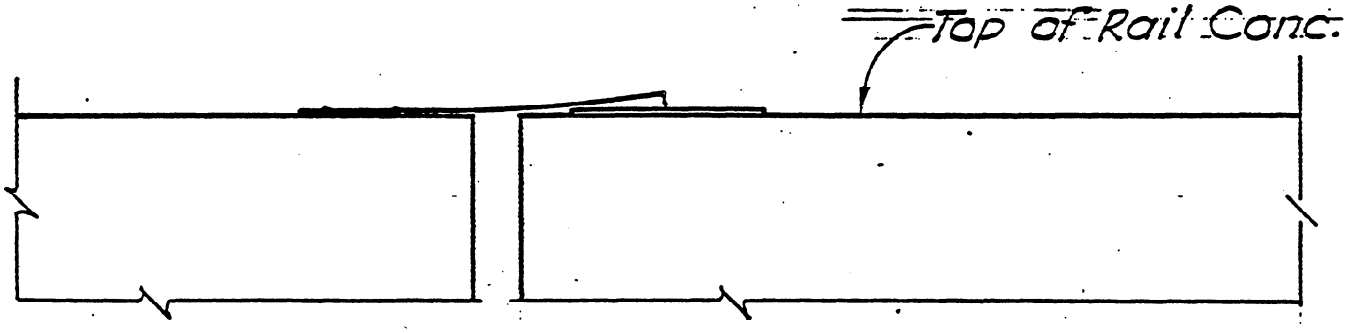
I (abut) ≈ $\frac{78}{12}(2.5)^3$

≈ 102

EXPANSION JOINT SCRIBE



PLAN



PART ELEVATION

NOTES:

1. Install one scribe at each deck joint on the most convenient side of the roadway i.e., widest shoulder. Use 8" scribe and 4" plate for joints having movement rating of 2" or less. Use 10" scribe and 6" plate for joints having movement rating greater than 2".
2. Place scribe on top of the concrete portion of the barrier railing.
3. Sand or wire brush surfaces of scribe and concrete to insure good adhesion.
4. Mix only enough epoxy for one scribe and plate when using the 681-80-44 Rapid Setting Epoxy. (5 min, pot life @ 70°)
5. Use weight on a piece of paper to hold the scribe down on the concrete surface while the epoxy is setting.
6. Mark the Initial Position of the scribe, date, and concrete temperature on the plate as shown with a scriber. Measure the concrete temperature by placing the bulb of a concrete thermometer 6" + into the deck section, if possible, or at any convenient location to obtain the approximate superstructure temperature.

SAMPLING AND TESTING OF TYPE "B" JOINT SEALS

The following revised instructions for sampling and testing Type "B" joint seals have been issued by the Transportation Laboratory. The procedures are currently in use. If there are any questions call Richard Spring at (916)739-2314.

1. Following the manufacturing of a given quantity of various sizes (Movement Ratings) of joint seal materials for use on Caltrans contracts, such as:

   MR=1" (1500 LF)
   MR=1 1/2" (1000 LF)
   MR=2" (1000 LF)

   The manufacturer will notify our Caltrans Laboratory (Richard Spring (916)739-2314).

2. Mr. Spring will arrange for an independent inspection agency to contact the manufacturer for the purpose of sampling the various lots of materials at the source.

3. The sampling agency will obtain one 3' long sample of each size and lot of material for every 500 LF 2 and send to our laboratory for testing along with the manufacturer's test report. The manufacturers lot number will appear along the length of the seal.

4. Following satisfactory testing, the manufacturer will be notified and the material will be set aside for <u>stock</u> to be used on Caltrans contracts only.

5. As the manufacturer receives orders and makes shipments to the individual contracts, form letters will be sent to our Caltrans Laboratory and with the shipment to the jobsite. This letter will contain the following information:

   a. Name and address where the seal is being sent.

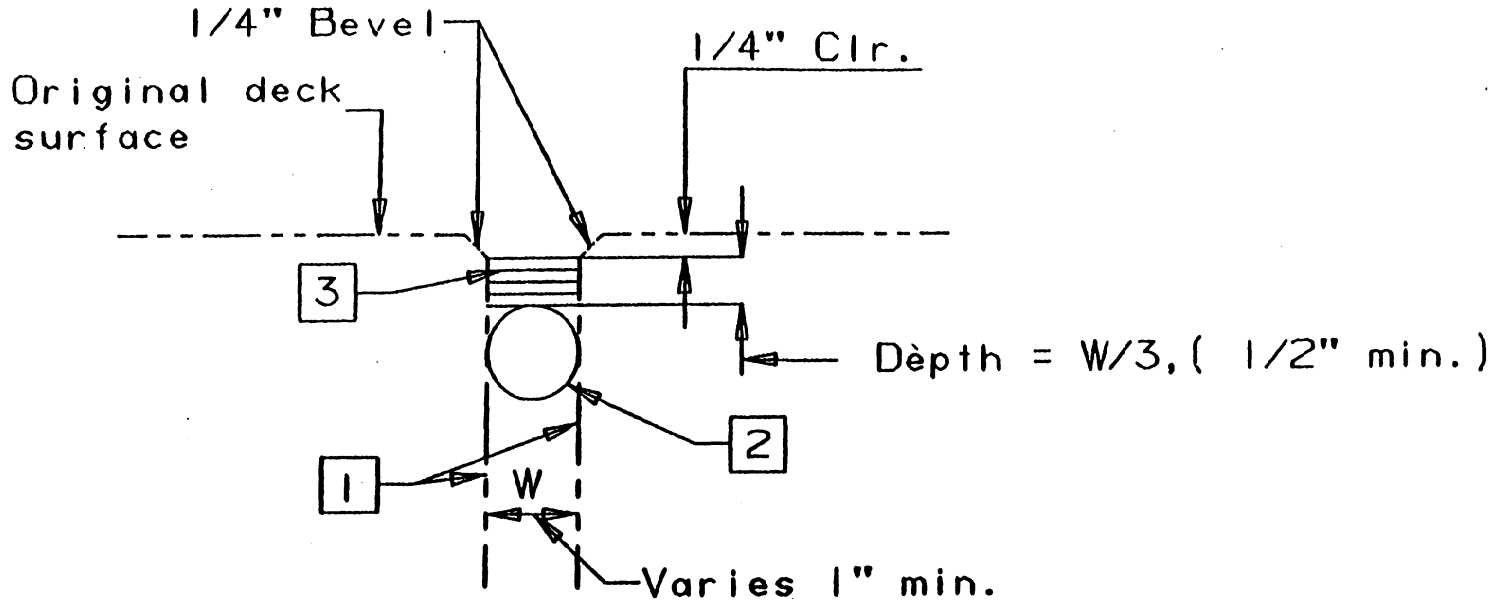
   b. State Contract Number.

   c. Size, quantity and movement rating of the seal.

   d. The Lot Number identifications.

   e. The TransLab's test number (SM number).

Upon receiving the letter from the supplier as to where the seal is being sent, the TransLab will send to the RE or Structure Rep a copy of the test report for the particular lot of material. Included on the test report will be the W_1 and W_2 values for the seal. The RE or Structure Rep should verify the lot number on the seal with the test report lot number.



JOINT SEAL TYPE A MODIFIED (MR 1/2")

No Scale

Notes:

1. If required, sawcut or grind transverse joints to the minimum (W) shown. Clean and abrasive blast joint.
2. Install commercial quality closed cell polyethylene rod stock with glazed surface. Diameter = joint width + 1/4".
3. Install joint seal. Place joint seal 3" up into curb or rail on low side of deck.

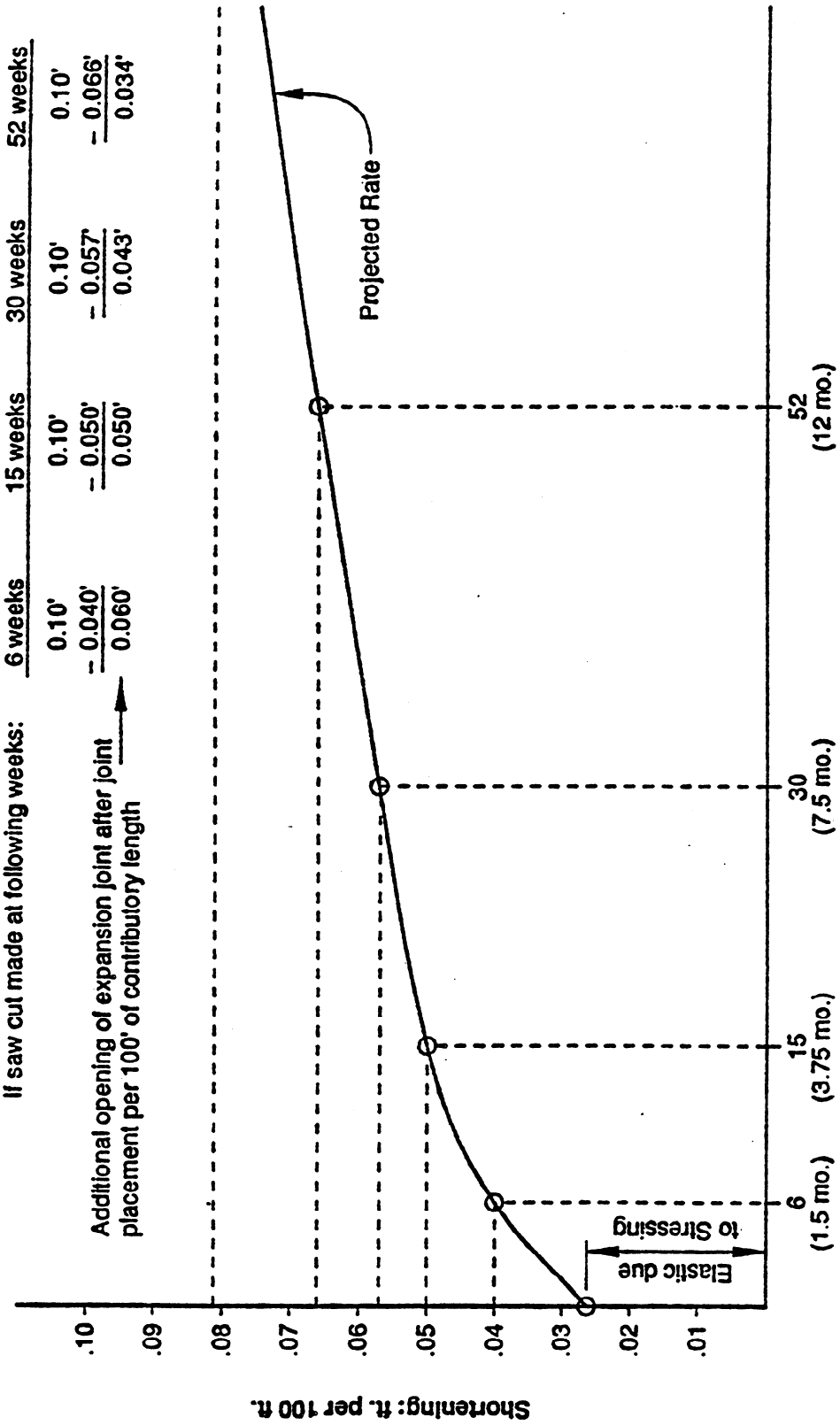
For details not shown see B6-21

Prestress Shortening

Assume long term total shortening is 0.10'/100'

If saw cut made at following weeks:

	6 weeks	15 weeks	30 weeks	52 weeks
	0.10'	0.10'	0.10'	0.10'
Additional opening of expansion joint after joint placement per 100' of contributory length	− 0.040'	− 0.050'	− 0.057'	− 0.066'
	0.060'	0.050'	0.043'	0.034'



BRIDGE CONSTRUCTION BULLETIN **Approved:**_____ **R. P. SOMMARIVA, Chief** **Division of Structure Construction**	**File: BCM 135-2.1** **MISCELLANEOUS CONSTRUCTION MATERIALS** **Date: January 20, 2000** **Expires: July 1, 2001** **Supersedes: None**

Subject: Release and Reporting of Values for Type B Joint Seal

The last paragraph of Sheet 2 of 7 of Bridge Construction Memo 135-2.0 is modified to read:

> Saw cutting shall not be started until the Type B seal material has been verified as having successfully been tested by the Division of Materials Engineering and Materials Testing Services (DMETS). The contractor/subcontractor will need to provide information to the Structure Representative regarding the manufacturer, lot number, date of manufacture and movement rating of the joint seal intended to be used prior to bringing the seal to the job site. The Structure Representative will verify the successful testing by contacting DMETS at 916.227.7263 and to obtain the W_1 and W_2 values for the lot of seal that will be used by the contractor. The movement rating (M.R.) ($W_1 - W_2$) of the Type B seal must be equal to or greater than that shown on the contract plans.

When contacting DMETS for W_1 and W_2 information, the Structure Representative (caller) should have the following information readily available:

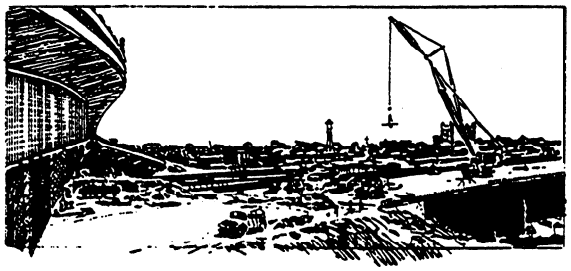
Manufacturer of the Type B Seal
Lot number shown on the side of the Type B Seal
Date of manufacture
Movement Rating for the seal

If requested, a copy of the DMETS test report for the Type B seal can be sent to the Structure Representative. It is important to note that the Type B Joint Seal will arrive at the jobsite without any state inspection release tags and no report of inspection document (TL-0029).

Background:

Type B joint seal is normally supplied by one of two sources from producers in the Midwest. As part of the manufacturing process, the various sizes of Type B Joint Seals are identified on the side of the seals with a Lot Number which represents a certain quantity. A sample of each lot of material is sent to DMETS in Sacramento for testing. Following the successful testing, the manufacturer is notified that the lot or lots of Type B Joint Seals are acceptable. At the time testing is performed, the lab does not know on which state contracts the Type B Seals will be used, therefore no information can be sent to the Structure Representative.

c: BCR&P Manual Holders
Consultant Firms
PStolarski, Chief Division of Materials Engineering and Testing Services
RWWolfe, Acting Chief Office of Structural Materials
BPieplow, Acting Construction Program Manager



COMPRESSIVE DEFLECTIONS OF FIBERGLASS OR STEEL REINFORCED ELASTOMERIC BEARING PADS

Elastomeric Bearing Pads consist of alternate layers of elastomer and steel sheet or fiberglass fabric reinforcement.

The compression deflections for steel and fiberglass reinforced elastomeric bearing pads can be reliably predicted within the normal range of construction tolerances. Transportation Laboratory tests have found that compressive strain is dependent upon two factors - compressive stress and shape factor. In addition, tests showed that compressive stress/strain behavior of fiberglass or steel reinforced pads is not significantly dependent upon overall pad thickness. In most situations the compressive deflection of the pad will be so small as to not effect the profile of the bridge. However, in the case of a hinge the magnitude of the deflection should be investigated, as it may be significant.

Attachment No. 1 of this Bridge Construction Memo shows two tables with families of curves which can be used to predict compressive deflection based on stress, shape factor, and strain. There is a separate table for fiberglass reinforced or steel reinforced pads which apply regardless of overall pad thickness. If long term compressive creep is to be included in the prediction, the strain values obtained from Attachment No. 1 should be increased by 25 percent.

Lab tests have shown that fiberglass and steel reinforced pads recover from dynamic creep caused by live loads. Therefore, dead load stress only will be considered in determining compressive deflection. Current bridge design practice limits the nominal compressive stress on a pad to 800 psi due to dead load and live load, not including impact. For steel reinforced pads with a shape factor $\geq$ 7.5, the average pressure shall not exceed 1000 psi, For calculating compressive deflection in the field, a dead load stress of 600 psi should be used. If a more accurate value of dead load stress is desired, contact the Structures Design Section responsible for your contract plans.

For special situations where extreme accuracy is desired, sample pads can be tested by the Transportation Lab to determine the stress/strain behavior of each lot of pads.

SAMPLE CALCULATION

Consider a 12"x18"x4" fiberglass reinforced bearing pad

Assume compressive stress of 600 psi

$$\text{Shape Factor} = \frac{\text{width x length}}{\text{width + length}} = \frac{12(18)}{12+18} = 7.20$$

From Attachment No. 1

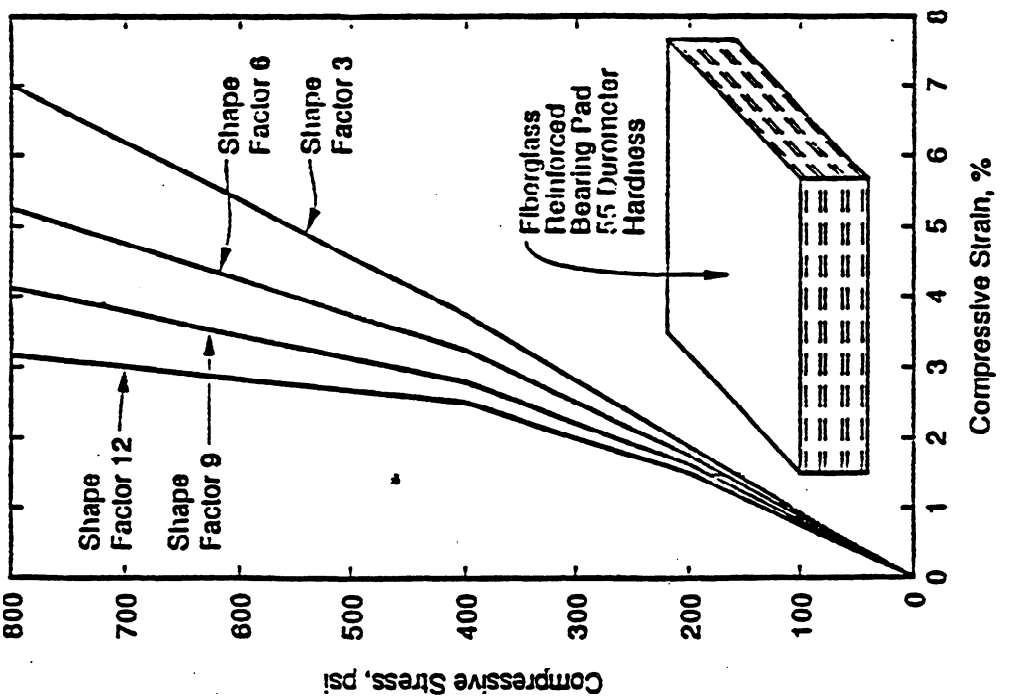
Strain = 4.0% = .040

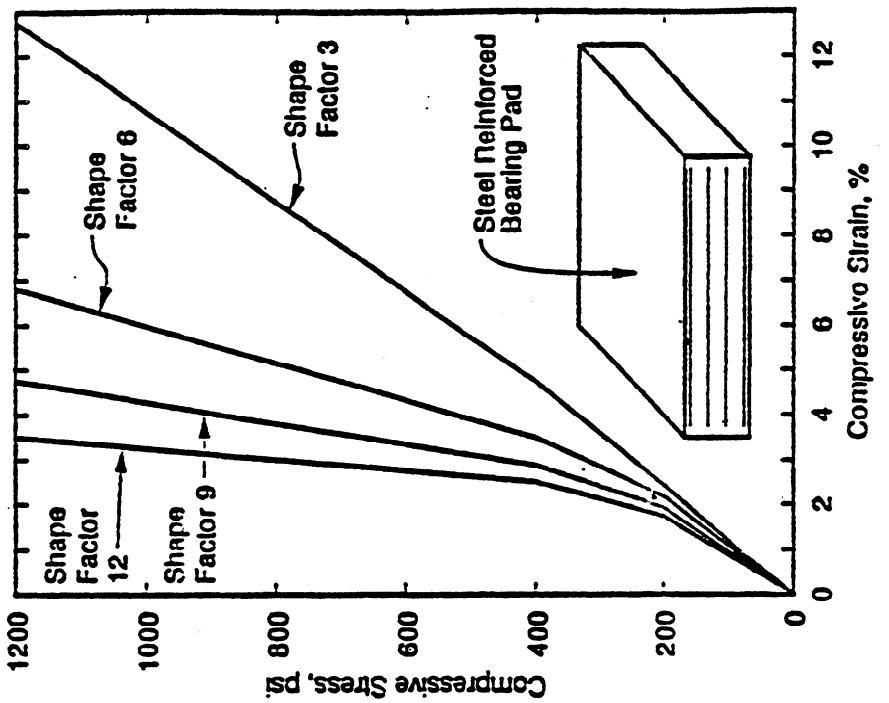
Compressive deflection including long term creep of 25% is equal to:

(Total Pad Thick.)	(Strain)	(125%)
4.00	(.040)	(1.25) = .20"

Say 3/16"

The bearing pad thickness shown on the plans will be that for fabric reinforced pads. Note that steel reinforced bearing pads are thicker than the corresponding fabric reinforced pads. If the thickness of a fabric reinforced pad is T (inches), the thickness of the corresponding steel reinforced bearing is 1.15 T. See Attachment No. 2 for a table of thicknesses for Steel Reinforced pads. For pads more than l/2 inch thick, it is the responsibility of the contractor to notify the Engineer in writing of the type of pad to be used. Bearing seat elevations must be set to correspond to the bearings to be used.


Compressive Stress, psi
Compressive Strain, %
Shape Factor 12
Shape Factor 9
Shape Factor 6
Shape Factor 3
Fiberglass Reinforced Bearing Pad 55 Durometer Hardness
0
100
200
300
400
500
600
700
800
0
1
2
3
4
5
6
7
8


Compressive Stress, psi
Compressive Strain, %
Shape Factor 12
Shape Factor 9
Shape Factor 6
Shape Factor 3
Steel Reinforced Bearing Pad
0
200
400
600
800
1000
1200
0
2
4
6
8
10
12

MEMO TO DESIGNERS:

Background

Our policy has been to standardize on l/2″ layers of elastomer. Until recently, we used very thin steel plates and a minimal elastomer cover at the top and bottom for the steel reinforced pads. The minimal thickness of cover and of steel, was ignored and the bearing-thickness shown on the plans was the sum of the l/2-inch thick layers. This resulted in a simple, standard Caltrans procedure for the design and manufacture of both the fabric reinforced and steel reinforced bearing pads.

The steel reinforcement option was removed from the 1981 Standard Specifications because the bearing manufacturers could not properly mold the bearing with the thin steel plates.

Current Policy

The current specifications for elastomeric bearings permit the use of the steel reinforced bearing as an option. However, the proper design of the steel reinforced bearings requires 14 gauge (0.075 inch) steel plates, full l/2-inch elastomer layers between the plates and a l/4-inch cover top and bottom. Therefore, the steel reinforced bearing will always be thicker than the corresponding fabric reinforced (fiberglass) bearing pad.

Design

There is no change in the design procedure. The designer will continue to design for the required number of l/2-inch layers and call out the thickness of the bearing as the sum of the l/2-inch thick layers. In permitting the use of the steel reinforced bearing as an option, the specifications require that the contractor notify the Resident Engineer of their choice. If the steel reinforced bearing is selected, the bearing seat elevation will be adjusted (lowered) by the Resident Engineer to allow for the increased thickness. The minor increase in compression on the steel plates due to the l/8" side cover may be ignored.

For most cast-in-place concrete, precast concrete and steel superstructures, there should be no difficulty in adjusting the bearing seat elevation at the time the contractor selects the bearing type. In general, there is no need for the designer to be concerned with the choice.

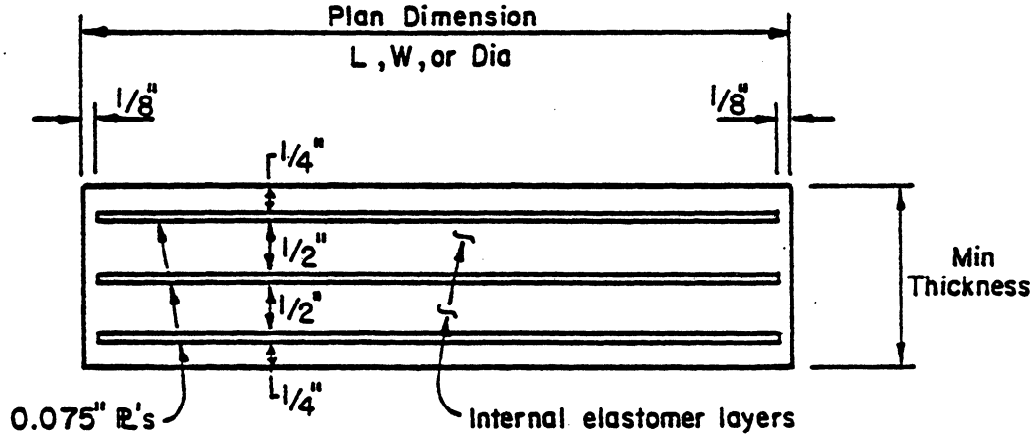
For some applications, the designer may want to limit the bearings to only one of the two types. If this is done, indicate the type selected on the plans as follows:

"Elastomeric Bearing Pads (fabric reinforced)" or,

"Elastomeric Bearing Pads (steel reinforced)"

Steel reinforced bearings are recommended where anchor bolt holes are required through the bearings. To assist the designer, the total thickness for steel reinforced bearings is tabulated below:

Design Thickness	Number of l/2" Layers	Number of Steel Plates	Actual Thickness Minumum	Maximum
1″	2	2	1.15	1.29
1.5″	3	3	1.73	1.89
2"	4	4	2.30	2.48
2.5″	5	5	2.88	3.08
3″	6	6	3.45	3.67
3.5″	7	7	4.03	4.27
4.0"	8	8	4.60	4.86



STEEL LAMINATED ELASTOMERIC BEARING

BRIDGE CONSTRUCTION MEMO 135-3-0
Attachment No. 2 (4-16-90)
Sheet 2 of 2

Volume II

MECHANICAL ANCHORAGE DEVICES

Expansion anchorage devices seldom develop the full tensile strength of a stud or a bolt, and are therefore less desirable than cast-in-place bolts and inserts. For this reason, expansion anchorage devices are generally used only for attaching fixtures such as signs, ladders, utilities, and temporary railings to hardened concrete.

Inspections by Structures Maintenance Engineers have disclosed instances of loose anchorage devices for bridge-mounted signs. The lack of proper anchorage had gone undected because a headed bolt was used instead of a threaded stud. When the bolt was tightened against the fixture mounting plate, it pulled the anchorage loose from the concrete. The anchorage then pressed against the other side of the plate, and further tightening gave the impression that the fixture was securely attached when actually it was loose.

In order to minimize this problem, plans call for a threaded stud instead of a headed bolt. The Standard Specifications require that the expansion anchor be recessed l/2" to 1" below the concrete surface after it has been expanded. This allows the inspector to observe if the anchorage has been seated initially, and if it is properly holding at the time the fixture is attached. The plans also call out the diameter of the stud. Galvanizing requirements are given in the specifications.

In the event that the aforementioned details are not shown on the project plans, the Structure Representative should insist that studs and nuts be used instead of bolts. Note that the stud diameter should be 5/8" if the plans call for a 5/8" anchorage device. The other aspects of the expansion anchorage shall conform to the project plans and specifications.

Note that when anchorages are expanded by driving the expansion element over an expander plug, a sufficient thickness of concrete must be provided behind the plug to resist the driving force. The drilled hole for the anchorage must also be true to size and shape so to assure the fullest bearing of the expanded anchor against the concrete.

All concrete anchorage devices shall be subject to the approval of the Engineer. Current approval lists can be found at http://www.dot.ca.gov/hq/esc/approved_products_list/. On the page are 'Cartridge Epoxies' and 'Mechanical Expansion Anchors' that link to the latest approved products. If the proposed MEA does not appear on the working list, approval shall be contingent upon the submittal to the Engineer of sample concrete anchorage devices, manufacturer's instructions, and certified results of tests indicating compliance with specification requirements.

In summary, the Structures Representative should be sure that all expansion anchorage installations conform to the following:

1. Be sure the anchorage device is listed on the approved working list (Website given).

2. Proper size hole is drilled.

3. Use threaded studs and not headed bolts.

4. Use galvanized studs. (Not black steel)

5. Be sure the expansion part of the anchorage is properly recessed below the surface after it has been expanded.

6. Never accept a stud of smaller diameter than the stud or anchorage device size called for on the plans.

Another useful resource can be found in the Bridge Design Aids. Pages 81 through 92 of Chapter 5, Concrete Design, show a properly installed anchorage and indicates where anchors may be used and what will be shown on contract plans. The complete manual is available on Structure Design's website at the following address:
http://onramp.dot.ca.gov/hq/esc/sdsee/design_technical_services/publications/bridge_design_aids.shtml

TABLE OF CONTENTS

MEMO NO.	ISSUE DATE	TITLE
145-1.0		(Blank)
145-2.0		(Blank)
145-3.0	05/04/1981	Support Systems for Portions of Permanent Bridges Which are Temporarily Unstable
145-4.0	12/01/1996	Contractor Designed Temporary Bridges or Other Temporary Facilities
145-5.0	10/11/1988	Bonding Dowels in Drilled Holes
145-6.0		(Blank)
145-7.0	11/19/1984	Adjustable Template for Checking Profiles of Ducts in Post-Tensioned, Prestressed, Concrete Girders
145-8.0	06/28/2006	Earth Retaining Structures
145-9.0	05/20/1987	Review of Working Drawings for Proprietary Earth Retaining Systems

Robert A. Stott, Deputy Division Chief
Offices of Structure Construction

**Denotes the document is a Bridge Construction Bulletin*

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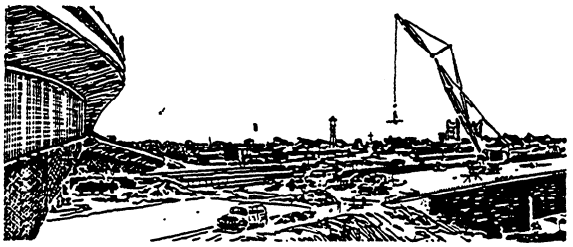


BRIDGE CONSTRUCTION MEMO 145-3.0

MISCELLANEOUS INFORMATION

May 4, 1981

Sheet 1 of 1

Volume II

SUPPORT SYSTEMS FOR PORTIONS OF PERMANENT BRIDGES WHICH ARE TEMPORARILY UNSTABLE

Occasionally portions of permanent bridges are unstable during some stages of construction. Examples of such unstable portions of bridges are sloping abutments, and bent columns, where these components are hinged at the footing and not yet stabilized by completion of the superstructure. Numerous other conditions of instability of portions of the permanent structure may occur during individual phases of construction.

It is essential that the Structure Representative determine if, when, and where such conditions of instability may occur. When it is determined that a portion of the permanent structure will be unstable, the Contractor should be required to submit working drawings showing details of his proposed temporary support system. (Section 5-1.02 of Standard Specifications) The Structure Representative shall review these working drawings to ascertain that the proposed support system is adequate to provide the necessary stability. It is especially important that these procedures be followed when there is an unstable portion of a bridge adjacent to a railroad or to an area occupied by public traffic.

Contractors frequently make use of wire rope "guys" to temporarily support unstable portions of bridges. The Structure Representative should be alert to the fact that a poorly designed wire rope "guy" system, or the improper installation of wire rope "guys", may result in a catastrophic failure of that portion of permanent structure that is being stabilized by the "guy" system.

For information concerning the proper use of wire rope, refer to the report prepared by John MacNeill entitled "The Use of Wire Rope Guys and Restrainers for Concrete Forms and Structural Components", dated May, 1975. This report is available to all registered Civil Engineers of the Office of Structure Construction.

Volume II

CONTRACTOR DESIGNED TEMPORARY BRIDGES OR OTHER TEMPORARY FACILITIES

If the Contractor proposes to construct a temporary bridge or other temporary facility across a traveled way being used by public traffic, the Contractor shall furnish the Structure Representative with working drawings and calculations detailing the location and design of the structure.

The Structure Representative will forward the aforementioned working drawings to the Sacramento Structure Construction Office for review by the Office of Structure Construction Falsework Section. The drawings should be forwarded with a letter describing loads to be supported, date Contractor wishes to begin construction, the design criteria used, and any other information pertinent to the review.

Temporary bridges or other temporary facilities not shown on the contract plans shall conform to Section 5-1.02, Plans and Working Drawings and Section 7-1.09, Public Safety, of the Standard Specifications.

Construction of the temporary bridge or other temporary facility should not be started until after the working drawings have been approved by the Engineer.

BONDING DOWELS IN DRILLED HOLES

There are three materials currently specified to bond a reinforcing dowel into a drilled hole in existing concrete.

Grout

Where plans or special provisions do not specify a bonding material a cement paste is to be used as outlined in Section 51-1.13 of the Standard Specifications. Special Provisions will contain a section "Drill and Grout Dowels where this is a separate contract item.

Magnesium Phosphate Concrete

Plans may indicate "Drill and Bond Dowel ... etc." Special Provisions will have a section "Drill and Bond Dowels" which refers to Section 83-2.02D(l) of the 1988 Standard Specifications. This specification is used where concrete barriers are to be constructed on existing bridge decks and can be used in other locations where shown on the plans. (Note - Corresponding 1984 Standard Specifications refer to Section 51-l.13 for bonding requirements.)

Epoxy

Plans in this case will indicate the use of epoxy bonding material. Special Provisions will have a section "Drill and Epoxy Bond Dowels" and there will be a separate contract item to cover the work.

The Structure Representative should not permit a substitution of the specified materials unless approved by the Structure designer.

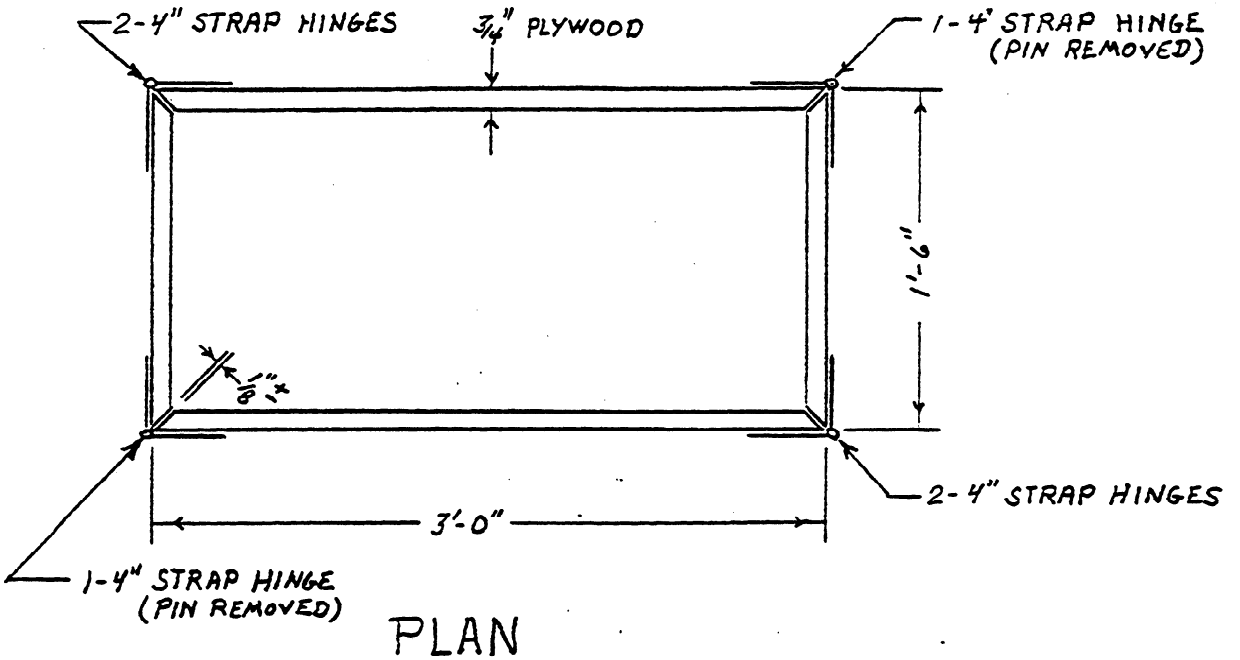
CONTAINER FOR USE IN PERFORMING THE BALL PENETRATION TEST

California Test 533 states that "The ball penetration test may be made on concrete in a wheelbarrow, buggy, or other container or after it has been deposited in the forms or on the subgrade." The test method requires that a minimum of three individual readings be taken for each penetration determination, and that the individual readings shall be at least nine inches between centers. Also, the minimum horizontal distance from the centerline of handle to the nearest edge of the level surface on which the test is made shall be six inches, and the depth of concrete above the bottom of the container or reinforcement shall be six inches for one inch maximum size aggregate and eight inches for larger size aggregate.

Attachment No. 1 to this Bridge Construction Memo is a drawing showing the details of a container which can be easily constructed and which will allow ball penetration tests to be made in conformance with California Test 533 requirements. As shown on the drawing, the container collapses and is therefore easy to store, transport, and handle. The container's size provides sufficient concrete to cast at least eight concrete test cylinders of the size required by California Test 540.

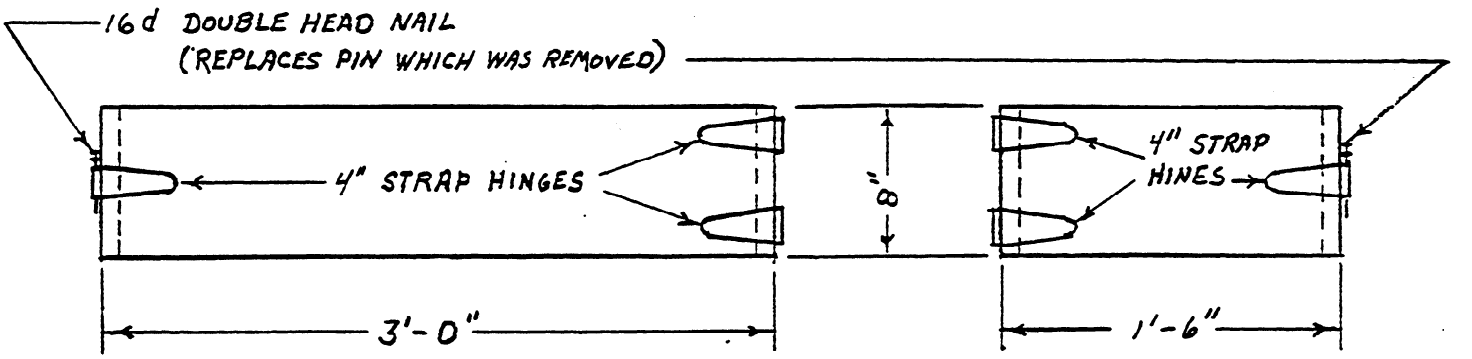
CONCRETE CONTAINER

FOR USE IN PERFORMING CALIFORNIA TEST METHOD 533



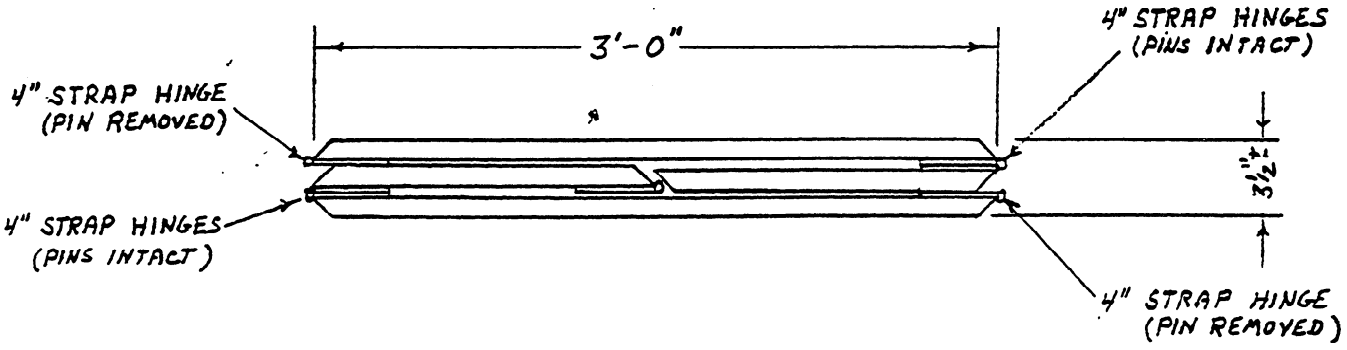
PLAN

NOTE: FOR DURABILITY, THE CONTAINER SHOULD BE TREATED WITH THOMPSON WATER SEAL OR A SIMILAR TYPE SEAL.

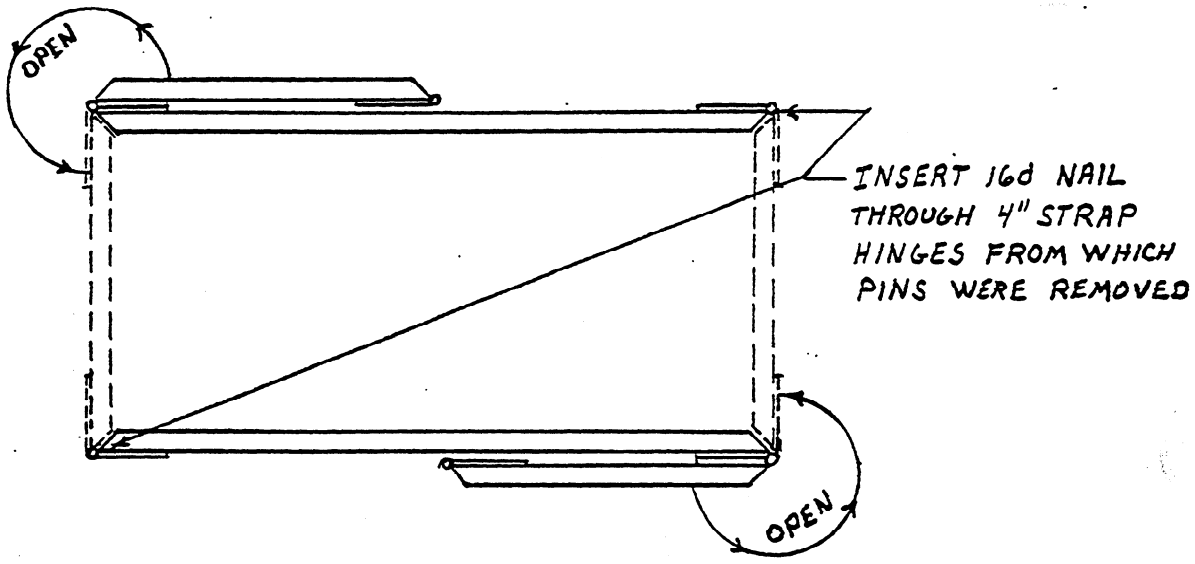


ELEVATION END VIEW

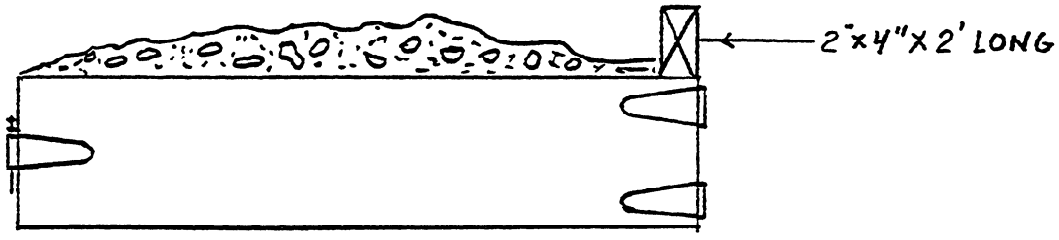
BRIDGE CONSTRUCTION MEMO 145-6.0
ATTACHMENT NO. 1 (12-28-81)
Sheet 1 of 2



CONCRETE CONTAINER IN COLLAPSED CONDITION

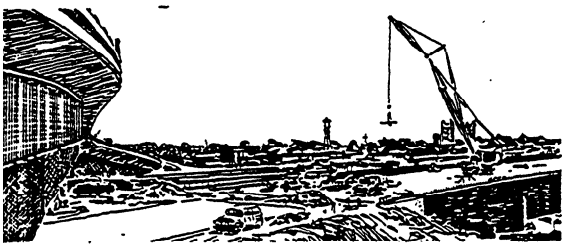


SETTING UP CONCRETE CONTAINER



CONTAINER FILLED WITH CONCRETE

NOTE: USE 2"x4"x2' LONG TO STRIKE OFF CONCRETE LEVEL WITH TOP OF CONTAINER PRIOR TO PERFORMING CALIFORNIA TEST METHOD 533



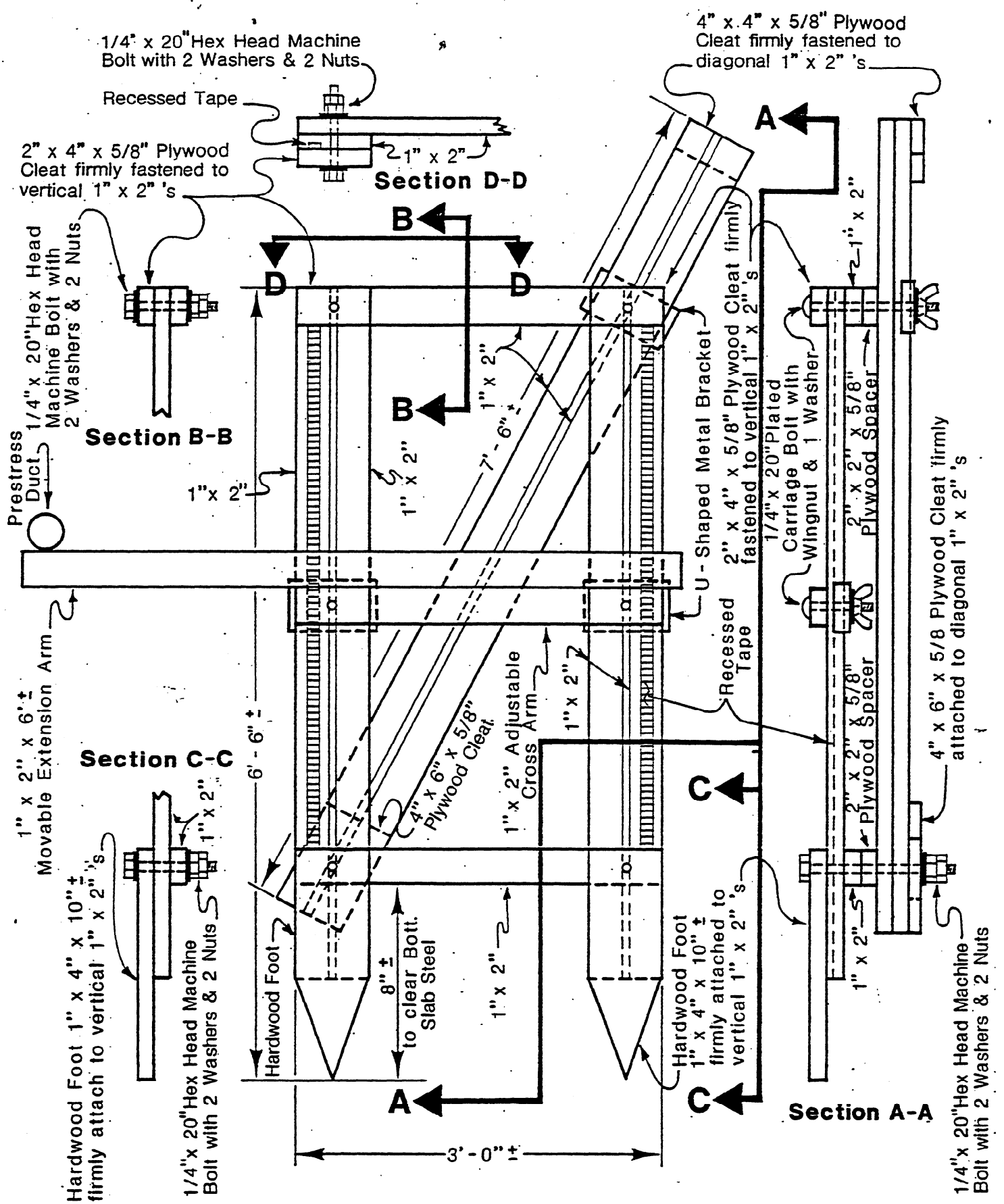
ADJUSTABLE TEMPLATE FOR CHECKING PROFILES OF DUCTS IN POST - TENSIONED, PRESTRESSED, CONCRETE GIRDERS

Attachment No. 1 to this Bridge Construction Memo is a sketch showing details for constructing a template for use in checking the profile grade of prestress ducts placed in post-tensioned, prestressed, concrete girders.

Attachment No. 2 to this Bridge Construction Memo describes the use of the template.

Although the template was designed primarily for use by the State's Engineer to check the duct profiles, it can also be used by the Contractor's ironworkers to set the ducts to the correct profile grade. It is understood that the use of the template can result in a considerable savings in time for contractor and state personnel, and reduce disagreements concerning the duct profile grade. The template is particularly useful for checking tendons in sloping exterior girders.

ADJUSTABLE TEMPLATE FOR CHECKING PRESTRESS DUCT PROFILES



BRIDGE CONSTRUCTION MEMO 145-7.0
ATTACHMENT NO. 1 (11-19-84)
Sheet 1 of 1

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INSTRUCTIONS FOR USING THE ADJUSTABLE TEMPLATE FOR CHECKING PROFILES OF DUCTS IN POST-TENSIONED, PRESTRESSED, CONCRETE GIRDERS.

1. Set the template on the soffit forms adjacent to the girder where the height of duct is to be established or checked.

2. Plumb the legs of the template and tighten the wingnut at the upper corner of the template.

3. Place the movable extension arm on top of the adjustable cross-arm and adjust the height of the cross-arm. The top of the movable extension arm should be set at the same height above the soffit as is required for the bottom of the duct.

4. When the top of the movable extension arm is at the correct height above the soffit, tighten the two wingnuts in the adjustable cross-arm. (Note that the adjustable cross-arm will be on the same slope as the bridge soffit.)

5. Slide the movable extension arm along the top of the adjustable cross-arm until it contacts the girder side form. The bottom of the duct will be at the proper height when it is set to contact the top of the movable extension arm.

BRIDGE CONSTRUCTION MEMO 145-8.0

MISCELLANEOUS INFORMATION

June 28, 2006

Volume II

MECHANICALLY STABILIZED EMBANKMENT WALL CONSTRUCTION CHECKLIST

There are several types of Mechanically Stabilized Embankment (MSE) wall systems. MSE walls all consist of three basic elements: select granular backfill, soil reinforcement (reinforcing strips or welded wire mesh), and precast concrete facing panels. A MSE wall is a composite system formed by the association of frictional soil backfill and soil reinforcement. Stresses produced within the soil mass are transferred by friction into the soil reinforcement. The soil reinforcement matrix is the key structural element of a MSE wall and can be either metallic or non-metallic (geosynthetic) depending on the system. The concrete facing distributes soil reinforcement loads, provides erosion protection, and remains flexible to tolerate differential settlement.

Each element of an MSE Wall can affect the overall performance of the structure. Diligent, timely inspection will help minimize common construction issues. The attached Mechanically Stabilized Embankment (MSE) wall construction checklist has been developed to serve as a resource and to assist field personnel with quality assurance inspection.

MECHANICALLY STABILIZED EMBANKMENT WALL
CONSTRUCTION CHECKLIST

I. SOURCES OF INFORMATION

- Bride Design Specifications (BDS) Section 5
- Bridge Design Aids – Chapter 3-1.1
- Caltrans Highway Design Manual – Chapter 200, Topic 210
- Earth Retaining Systems Committee web page at http://onramp.dot.ca.gov/hq/esc/sd/bridge_design/ers/

II. SPECIFICATION REQUIREMENTS:

- Contract Special Provisions (Recently amended as of 11/05, see page 5 for a summary of changes)
- Contract Plans
- Standard Specifications

1. Plans and Working Drawings, Section 5-1.02
2. Certificate of Compliances, Section 6-1.07
3. Earthwork, Section 19
4. Prestressing Steel, Section 50-1.05
5. Concrete Structures, Section 51
6. Elastomeric Bearing Pad, Section 51-1.12H
7. Mortar, Section 51-1.135
8. Reinforcement, Section 52
9. Corrugated Metal Pipe, Section 66
10. Underdrains, Section 68-1
11. Permeable Materials, Section 68-1.025
12. Galvanizing, Section 75-1.05
13. Engineering Fabrics, Section 88
14. Portland Cement, Section 90
15. Water, Section 90-2.03
16. Minor Concrete, Section 90-10

III. ITEMS TO BE RECORDED IN THE JOB FILES

- Reports of Inspection of Materials (Category 41)

    1. Soil Reinforcement
    2. Precast facing panels
    3. Elastomeric bearing pads
    4. Structure backfill materials
    5. Engineering filter fabrics
    6. Geodrain materials, if used

- Contractor's Submittals (Category 12)

    1. Shop Drawings (including manufactures installation manual)
    2. Excavation Plans, if needed
    3. Test data, if required (i.e. backfill, soil reinforcement couplers)

IV. PROPRIETARY EARTH RETAINING SYSTEMS:

The contract Special Provisions usually gives the contractor the option of choosing one of the proprietary systems listed in the Department's current list of pre-qualified earth retaining systems. These systems are alternatives to the fully detailed State system shown on the contract plans. The pre-qualified list can be accessed from the OSC home page under "Field Resources."

V. CONSTRUCTION

Before Construction:

1. Review Construction Considerations Section of Foundation Report. Contact Report author if there are any questions or unusual geotechnical requirements such as settlement monitoring, surcharge, etc.
2. Prior to beginning work set up a pre-construction meeting with your contractor to discuss MSE wall construction. Invite representatives from METS, OSD, Geotechnical Services, MSE wall specialist, District Construction and Materials Lab.
3. Remind the Contractor to submit wall type and working drawings to the Structure Design - Document Units in Sacramento. Only one type of wall will be used at each wall location in a project. Working drawings should include manufacturer installation manual.
4. Ensure the Contractor verifies the field dimensions and elevations prior to preparing the working drawings. This will mitigate any potential problems that may arise due to the precast panels being too short or too tall.
5. Remind the Contractor to submit the construction staking request for the wall layout line, beginning of wall, end of wall and respective elevations.
6. Review shop drawings and verify the top of wall elevations versus the district grid grades, drainage plans and underground utility plans. Verify the locations of the soil reinforcement to ensure no conflict with any utility locations.
7. Coordinate with the Structure Designer on review and approval of MSE wall type. When the submitted shop drawings are approved with minor corrections, the Structure Representative is responsible to assure the Contractor incorporates such details in the final construction.
8. Confirm with the Contractor on material procurement, sampling and scheduling.
9. Review and approve the final texture of the sample facing panel and concrete mix design. The sample panel shall be stored for later comparison to the finished product.
10. Review and approve the erection procedures.
11. Review and approve excavation plan, if required.

During Construction:

1. Check sloping or shoring installation if required.
2. Ensure the leveling pad is constructed properly. Check against the reference stakes for location and elevations. An improperly placed leveling pad can result in subsequent panel misalignment and decrease wall construction productivity.
3. All materials incorporated into the work must be inspected thoroughly prior to placement. Once constructed, it is difficult to remove and replace. Check concrete panels for cracks, surface irregularities, and dimensions. Check for damage to the galvanized coating on the metallic reinforcement.
4. Ensure proper storage handling. Geosynthetic reinforcement, polyurethane foam, and filter fabric must be stored away from sunlight.
5. Collect Certificates of Compliance and any samples as required. Take photos of release tags that are stenciled on the backside of the panel.
6. Backfill material shall be sampled and tested for gradation and property requirements listed in the Contract Special Provisions. The District Materials Laboratory typically performs gradation, PI, and SE tests. Corrosive tests (i.e., chlorides, sulfates, and resistivity) are performed by the Sacramento METS Laboratory.
7. Ensure the Contractor provides adequate equipment to lift and set panels.
8. Ensure the wall LOL is straight and at the right batter, as specified in the shop drawings, to account for the backfill displacement. Verify that the vertical and horizontal tolerances, specified in the Contract Special Provisions, are met during wall construction.
9. Ensure all joints are protected with engineering filter fabrics prior to backfilling.

10. Ensure the Contractor installs drainage system, if required per plan before backfilling.
11. Ensure backfilling is done with proper equipment to avoid damage to the panels.
12. Sheepsfoot rollers are not to be used for compaction of select granular backfill within the limits of the soil reinforcement. Handheld or hand-guided compacting equipment shall be used within three feet of the facing panels.
13. Space bars must be placed between two adjacent panels to maintain the spacing of the interlocked panels.
14. The number of soil reinforcements per panel may vary throughout the length of the wall. Ensure the Contractor places the panels correctly. It is of utmost importance that the panel type and number of tie strips in the panel coincide with the requirements shown on the approved plans.
15. Reinforcing strips must be placed in the proper alignment and on the level surface, usually an inch higher to avoid down drag by the backfill as a result of compaction. Ensure the locations of test strips are centered in the opening of the panels.
16. Check condition of the soil reinforcement prior to backfilling.
17. Check that all test strips are in place according to the contract plans.
18. Reinforcing strips shall be installed perpendicular to the back of the panel, unless otherwise noted.
19. Once the system is complete, a concrete cap will be formed and placed. Ensure all reinforcing steel for the barrier rail is placed before concrete placement.
20. If a significant rain event is anticipated, at the end of each shift the contractor shall grade the backfill to slope away from the wall face to keep surface run off away from the panels. Surface water if not controlled will migrate the fines from the structure backfill into the pervious material, possibly causing excessive backfill settlement.
21. Check wall cap profile elevations to match the cross section and super elevations of the roadway section. Do not allow contractor to drive equipment directly on geosynthetic material or metallic reinforcement.
22. Send a 60-pound sample of backfill material to METS in Sacramento for each level where inspection elements are installed.
23. Send one 5-foot long representative sample of the soil reinforcement material to METS in Sacramento for testing.
24. If the soil reinforcement material is galvanized, measure and record the thickness of the zinc coating along the length of the least of 3 or the total number of permanent reinforcing elements at each level. Galvanized thickness can be measured with a magnetic thickness gage (PosiTector).
25. Obtain certified mill test reports and a Certificate of Compliance for the soil reinforcement.

VI. POTENTIAL PROBLEMS

OBSERVATION	POTENTIAL CAUSE
Wall Distortion: • Differential settlement that causes panels to contact each other resulting in chipping or spalling • Low spot in the wall profile • Overall wall leaning	▪ Weak or improper bearing material. ▪ Inadequate compaction and/or poor quality foundation material l ▪ Leveling pad not constructed per the tolerances specified in the approved shop plans.
Wall Leaning Out (away from backfill)	▪ Panels not battered sufficiently ▪ Large backfill placing and/or compaction equipment working within the 3' zone from the back of the wall ▪ Backfill material dumped too close to the free end of reinforcing strips, then spread towards back of wall, causing bulge in strips and pushing panels out ▪ Backfill material too wet ▪ Backfill contains excessive fines materials (beyond the Specifications for percent of materials passing a No. 200 sieve) ▪ Backfill contains excessive fine materials (beyond the Specifications

OBSERVATION	POTENTIAL CAUSE
	▪ Backfill material pushed against back of wall before being compacted on the strips ▪ Excessive or vibratory compaction on uniform fine sand (more than 60 % passing a No. 40 sieve) ▪ Wedges not seated securely ▪ Excessive lift thickness ▪ Clamps not tight ▪ Plasticity Index of backfill material in excess of the specification limit.
Wall Leaning In (towards backfill)	▪ Excessive batter set in panels for select granular backfill material being used. ▪ Inadequate compaction of backfill ▪ Panesl excessively battered ▪ Improper compaction of lower vackfill levels ▪ Settlement of the original ground behind the wall
Localized differential distortion between adjacent panels that causes points of inflection and excessively wide joints.	▪ Adjacent panels set at different battered angles.

V. FINAL RECORDS

After the completion of the MSE wall, the Structure Representative must send of copy of the following to the Headquarters Office of Structure Construction in Sacramento.

- Report of Completion
- Soil reinforcement data (attach to the Report of Completion)
    1. Certificate of compliance
    2. Mill Test Reports
    3. Measured Galvanized coating thickness

- As-Built Contract Plans

In addition to the as-built contract plans the Structure Representative must review the final shop drawings for correctness prior to having the contractor submit them to the Documents Unit in Sacramento. The specifications require that the contractor submit a final set of as-built prints showing all corrections to the Documents Unit in Sacramento. The Structure Representative must verify the final shop drawings were transmitted to the Documents Unit prior to authorizing final payment

VI. SUMMARY OF RECENT CHANGES TO THE MSE STANDARD SPEACIAL PROVISIONS

All contracts advertised after 11/2005 should contain a new version of the MSE Wall Standard Special Provisions. The major changes of this amended version are listed below:

Provisions have been made for geosynthetic (non-metal) soil reinforcement. Gradation and other backfill property requirements differ depending on the type of soil reinforcement used (e.g. geosynthetic or metallic).

Gradation and backfill soil properties requirements, for geosynthetic reinforcement, have been added to the MSE standard special provisions

Backfill for metallic soil reinforcement have been amended.
- Reduced PI requirement
- Reduced chloride requirement.
- Increased resistivity requirement.
- Reduced sulfates requirement.

Permeable material requirements for metallic soil reinforcement have been amended.
- Reduced chlorides requirement.
- Reduced sulfates requirement.

Permeable material requirements for geosynthetic soil reinforcement have been added.

Sampling and testing requirements for metallic soil reinforcement couplers were added. Note that these requirements do not apply to the proprietary systems.

Earthwork requirements were amended.
At the end of each work shift, the contractor must slope the grade away from the wall to prevent surface runoff from contacting the wall.
- A minimum of 95% relative compaction is required at all locations.

Horizontal and vertical alignment tolerances are now specified for the MSE wall.

Certificate of Compliance is now required for all MSE materials

Qualified representative of the proprietary MSE system is required to be present during construction of the first 3 meters of the entire wall. This person cannot be an employee of the Contractor.

Volume II

REVIEW OF WORKING DRAWINGS FOR PROPRIETARY EARTH RETAINING SYSTEMS

When earth retaining systems are to be constructed the Contractor is allowed the option of a State designed system which is completely detailed on the plans or a pre-approved proprietary system which will be listed in the Special Provision.

Memo to Designers 5-16, which is Attachment No. 1 to this memo, covers procedures for plan submittal and checking of working drawings for the proprietary systems. In that memo a reference to Memo to Designers 11-l is made. It can be found in Volume II of the Bridge Construction Records and Procedures in Memo 160-6.0.

The Special Provisions require submittal of 4 sets of plans from the Contractor to Structure Design. Two unchecked sets will be sent to the Structure Representative. One set shall be given to the Resident Engineer. It is important that any field changes in alignment, grade, earthwork, traffic handling, etc. which may affect the wall design be reported to design as soon as possible prior to final approval.

After plans are approved 3 sets will be sent to the Structure Representative. He will give one set to the Contractor's Field Representative.

Starting about July 1987 special -provisions will require the Contractor to submit proprietary wall plan tracings to Structure Design upon receiving notice of plan approval. As Built plans will be printed and sent to the Structure Representative who will record changes and send to design at the completion of the job along with other As Builts.

During construction, the proprietary wall shall be built using the approved plans and sheets or details of the State designed system which are referred to by the approved plans. In general the State designed plans are superseded by the proprietary wall plans except for those sheets or details which are used by reference.

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BRIDGE CONSTRUCTION MEMO 150-0.0

OVERLOADS

June 21, 2007

Volume II

TABLE OF CONTENTS FOR SECTION NO. 150

Memo No.	Date	Title
150-1.0	06-21-07	WEIGHT OVERLOAD GUIDELINES FOR BRIDGES ON CONSTRUCTION PROJECTS

DOLORES M. VALLS, Deputy Division Chief
Offices of Structure Construction

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Volume II

WEIGHT OVERLOAD GUIDELINES FOR BRIDGES ON CONSTRUCTION PROJECTS

Introduction

This memo provides the Offices of Structure Construction guidelines on the administration of the "weight limitation" provisions of a construction contract. These guidelines are to ensure uniform review and proper allowance for movement of construction equipment over structures not open to public within the limits of the construction contract. Overload cases that vary from these guidelines need to be forwarded to the HQ Office of Structure Construction in Sacramento for further analysis per the following procedures.

Standard Specification Weight Limitations

Section 7-1.02, "Load Limitations," of the Standard Specifications sets forth weight limitations for earthmovers, truck, and truck and trailer combinations that will be permitted to cross-completed bridge structures, and in addition, provides that other construction equipment may be permitted to cross bridge structures subject to the weight limitations and conditions of the California Department of Transportation Permit Policy. The provisions of Section 7-1.02 apply only within the limits of highway construction contracts.

The California Vehicle Code governs operation of vehicles (including construction equipment) on State highways beyond the limits of a construction project.

Overloads

When analyzing overloads, consideration should be made for the potential reduced capacity of a partially completed or demolished structure. Overloads on bridge structures within construction contracts may be either repetitive or occasional, dynamic (moving) or stationary. Listed below are guidelines for evaluating common overloads.

A. Repetitive Overloads

Repetitive overloads usually occur in connection with an earthmoving operation, and thus usually involve earthmoving equipment.

1. Bridge Structures Designed for HS20-44, Alternative, and Permit Live Loading

Section 7-1.02, "Load Limitations" will apply and the use of earthmoving equipment that exceeds these limitations will not be permitted.

2. Structures Designed for Overloads

Under the provisions of Standard Specification Section 7-1.02, "Load Limitations" the Contractor may request the redesign of a structure to increase its load carrying capacity to accommodate heavy construction vehicles such as earthmoving equipment.

At the present time, the design vehicle used to represent the construction equipment loading is a three-axle vehicle having a maximum axle load of 130 kips and a total gross load of 330 kips for spans greater than 54 feet, and a two-axle vehicle having a maximum axle load of 130 kips and a total gross load of 200 kips for spans of 24 to 54 feet. For spans under 24 feet, the design is based on a single 130 kip axle.

If a structure is designed to accommodate the 330 kip construction equipment loading, no further increase in load-carrying capacity will be considered and the use of construction equipment that exceeds the 330 kip loading will not be allowed.

If the stresses produced by the Contractor's equipment do not exceed the stresses produced by the aforementioned construction equipment loading (design vehicle) and if the Contractor is willing to pay the cost of redesign and the increased cost of construction, the structure or structures will be redesigned. (Additional information relative to construction overload design is given in Memo to Designers 15-15.)

Following are the procedures to be followed when the Contractor requests a redesign of a structure, or structures, to increase the load carrying capacity.

(i) Contractor submits a letter to the Resident Engineer requesting that the structure be designed to increase its load carrying capacity. In this letter the Contractor must name the structure or structures to be redesigned, give specific details of the loads and the positioning of the loads on the structure. The contractor must also state that they are willing to pay the cost of redesign and the increased cost of construction.

(ii) Structure Representative submits a copy of the Contractor's letter to the Offices of Structure Construction subdivision Deputy along with a request that the structure be redesigned. The Structure Representative should also request that they be advised of the estimated cost of redesigning the structure. At this point, the contractor should be informed of the estimated cost and a formal agreement should be reached prior to proceeding.

(iii) After the redesign has been completed and upon receiving revised contract documents and the estimated maximum cost of redesigning the structure, the Structure Representative will prepare a Contract Change Order. The Contract Change order will authorize the structural alterations to accommodate the construction overloads. If the final cost to the Contractor for the redesign is known then the credit to the State should be included. Otherwise a supplemental Contract Change order should be written when the final costs are completed. (See page 6 of this Construction Memo for a sample of this type of Contract Change Order.)

B. <u>Occasional Overloads</u>

Occasional overloads will include the movement of construction equipment (concrete trucks, cranes, paving equipment, etc.) across completed or partially completed structures from one

work site to another. Also included for consideration are stationary overloads, such as cranes, winches, and concrete pumps operating from the bridge deck.

1. Concrete Trucks

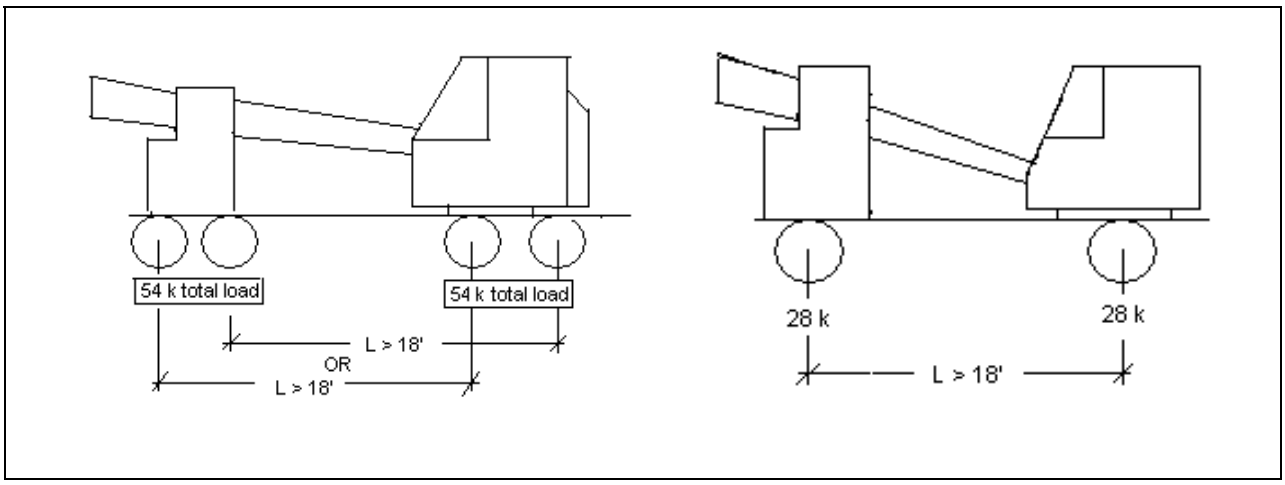
Concrete trucks traveling on the highway with full loads generally need to use booster axles to meet the axle weight requirements of the California Vehicle Code. When discharging concrete, the booster wheels need to be raised which increases the loads on the remaining axles resulting in axle loads that exceed the legal load allowed by the Permit Policy. Standard Specification Section 7-1.02 "Load Limitations," allows trucks over legal (exceeding CVC weight limitations) limit on bridges with up to 28,000 pounds for single axles and 48,000 pounds for the tandem axles. This limits most trucks to hauling a maximum 7 1/2 to 8 cubic yards. These trucks should be weighed to confirm allowable specification loading.

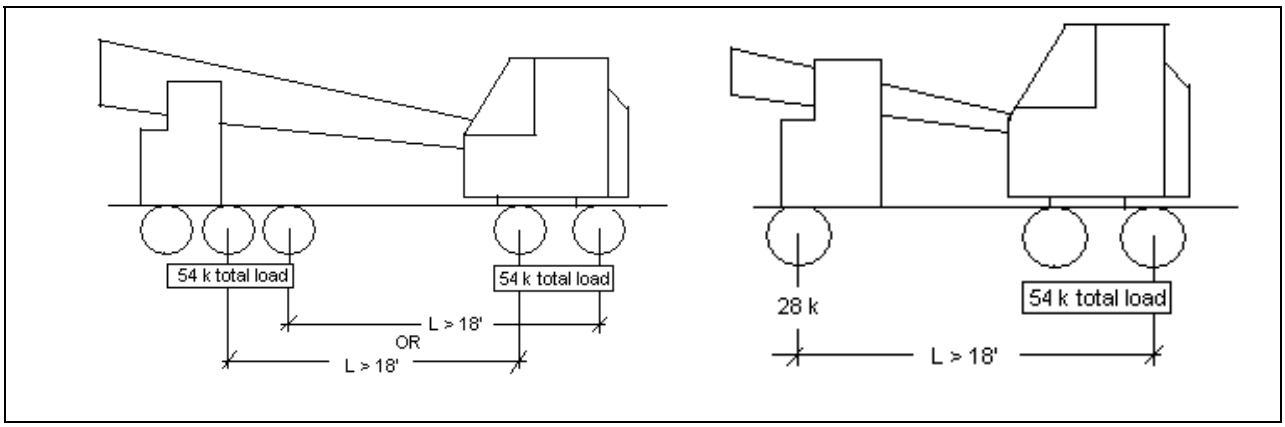
2 Cranes and Concrete Pumps

Fully equipped truck cranes are permitted to cross bridge structures on construction projects provided they conform to Permit Policy. The following general guidelines may be used to determine if truck crane or concrete pumper traveling on the bridge meets permit policy.

- Tandem axle weights less than 54,000 pounds
- Single axle weight less than 28,000 pounds
- No group of three axles within an 18 foot distance (see table diagrams below)

Any exception to the guidelines should be referred to the HQ Office of Structure Construction in Sacramento. The HQ Office of Structure Construction will forward the necessary information to Structures Maintenance and Investigations for decision. A letter requesting overloads analysis and a complete description of the vehicle should be included. (See Processing Requests to Move Overloads below for further details.)





Cranes are often stripped down (counterweights and other components removed) and sometimes the boom is supported on a trailer in order to achieve allowable permit weights. The Engineer should verify that the crane is configured in its traveling condition when moving on the bridge.

3. Stationary Cranes and Concrete Pumps

Cranes are also used in a stationary position to do work from bridges, including pile driving, lowering falsework, and lifting girders. Cranes lifting in a stationary position cause high outrigger loads. Outrigger loads greater than 40,000 pounds should be referred to the Sacramento Office of Structure Construction. The HQ Office of Structure Construction will forward the necessary information to Structure Maintenance and Investigations for decision. The engineer may review proposals for outrigger loads less than 40,000 pounds provided that the bridge is designed for permit loads or has full permit capacity (PPPPP). The Contractor should be required to provide calculations for outrigger loads. Outrigger loads may be distributed in one of three methods:

i. Outriggers that produce loads less than 25,000 pounds may be placed on timber mats. The mats should be 12 by 12 minimum and placed parallel to the girders. The minimum length of the mat is 5 feet.
ii. Outriggers that produce loads greater than 25,000 pound should be placed on a beam that distributes load fairly equally to two girders.
iii. Outriggers placed upon concrete bent caps of box girder bridges do not require mats or beams to distribute loads.

Submittals for stationary loading to be referred to Sacramento should include the following information:

i. Location of crane outriggers tied into reference locations (CL bent or abutment, CL bridge, or edge of deck etc.
ii. Calculations for outrigger loads.
iii. Manufacturers information for crane and a description of how the crane will be outfitted and configured (boom length and counterweights).
iv. Weight of what will be lifted and maximum extension of boom.
v. Proposed method for distribution of outrigger loads.
vi. How the configured crane will be moved into position while complying with Standard Specification Section 7-1.02, “Load Limitations”.

4. Track Equipment

Equipment such as pavement grinders and excavators occasionally need to cross or work on a bridge. The engineer with bridges designed for HS-20 loading may approve this equipment, provided that it meets the following conditions:

i. maximum gross weight less than 60,000 pounds
ii. maximum load on track less than 5 kips per foot

All other requests should be referred to the HQ Office of Structure Construction in Sacramento.

Processing Requests to Move Overloads

As previously noted, requests from Contractors to utilize earthmoving equipment or to move overloads which do not exceed the limitations of Section 7-1.02, "Load Limitations" of the Standard Specifications may be approved at the job level. All other requests are to be forwarded by the Structure Representative to the Sacramento Office of Structure Construction. The Office of Structure Construction will forward the necessary information to Structures Maintenance and Investigations for decision.

The Contractor's request must be explicit as to the nature of the overload and the conditions under which it will be moved. A complete description of the vehicle is required. The information required includes the type, make and model of equipment, the axle spacing, axle width out to out of tires, the axle load (obtained by scale weight if possible), the width and number of tires, operating conditions, etc.

Permission to cross bridge structures with construction equipment (including earthmoving equipment) that does NOT exceed the limitations of Section 7-1.02 will also be granted by means of a letter to the Contractor from the Resident Engineer (see page 6 for and example letter). However, if special conditions or limitations are to be imposed, they should be incorporated into a letter similar to the example letter authorizing the use of earthmoving equipment.

Since construction overloads will often affect areas of responsibility of both the District and the Offices of Structure Construction it is important that both be fully informed. Particular care should be taken by the Structure Representative to ensure that copies of all correspondence related to overloads are furnished to interested District personnel.

EXAMPLE CHANGE ORDER TO AUTHORIZE BRIDGE REDESIGN

As provided in Section 7-1.02 of the Standard Specifications, modify substructure of the Van Koevering Avenue Undercrossing, Bridge 54-1001, as shown on Sheets 2 and 3 of this change order to accommodate construction overloads.

It is agreed that the contractor will furnish all labor, equipment and material and perform all work required to accomplish the structural alterations at no cost to the State.

It is further agreed that the State will be credited by means of a supplemental Contract cost change order the actual cost for redesigning the Van Koevering Avenue Undercrossing to accommodate construction overloads. The design costs shall be a maximum of $10,000.

EXAMPLE LETTER TO AUTHORIZE THE USE OF EARTHMOVING EQUIPMENT

To Contractor

Gentlemen:

Your request dated (date) for permission to cross the (name of bridge), Br. No. (xxx), with construction overloads is approved in accordance with the provisions of Section 7-1.02, "Load Limitations" of the Standard Specifications, subject to the following conditions:

1. The approaches at each end of the bridge shall be completed to the grade required to provide a smooth transition to the bridge roadway, and shall be maintained in a smooth and uniform condition at all times while construction equipment is in use, for a length of not less that 150 feet measured from the bridge ends. Local depressions in the approaches in the vicinity of the bridge ends will not be permitted.

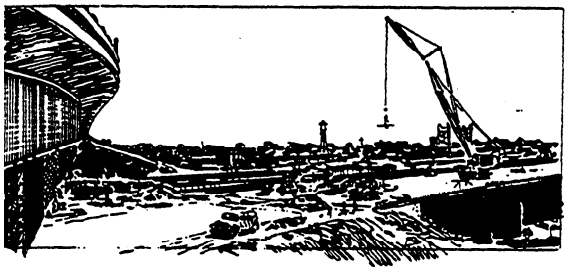
2. Construction equipment, either loaded or unloaded, shall be operated at all times at a speed and in a manner so that or bouncing of the equipment will not occur while the equipment is crossing the bridge.

3. Construction equipment shall be confined to the construction equipment lane by means of substantial, temporary physical barriers.

4. Only one construction overload will be permitted on the bridge at any time.

5. On completion of the operation that requires the use of a construction overload, the bridge roadway shall be cleaned and physical barriers used in connection with the construction equipment lane shall be removed and disposed of away from the job site.

   Note: Other conditions or restrictions may be added as necessary to suit particular job circumstances.



BRIDGE CONSTRUCTION MEMO 155-0.0

PAINT

April 2, 1990

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A. P. Bezzone

A. P. BEZZONE, Chief
Office of Structure Construction

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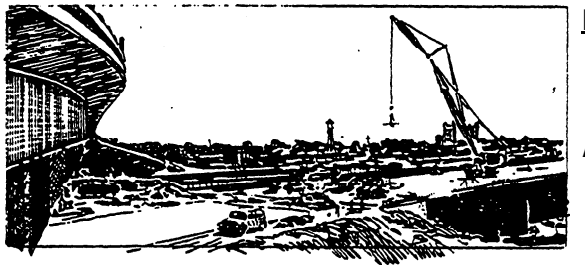

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CLEANING AND PAINTING OF STRUCTURAL STEEL

General Information

The cleaning and painting of structural steel bridges is a vital, specialized, and often controversial phase of bridge construction and maintenance.

Structure Representatives are responsible for the satisfactory completion of cleaning and painting work in accordance with the contract specifications, Attachment No. 3 to this Bridge Construction Memo is a check list which may be used to aid the Structure Representative in obtaining a satisfactory painting project.

The following information on cleaning and painting methods, procedures and precautions, paint material, inspection techniques and record keeping is intended to provide the Structure Representative's and their assistants with a rudimentary knowledge of the cleaning and painting work. Of course, any specific instructions in the contract specifications will supersede or modify these instructions.

Any special problems with regard to cleaning and painting, which cannot be solved by the Structure Representative, should be referred to Office of Structure Construction.

Purpose of Painting

The paint on structural steel may be described as a relatively impervious barrier imposed between the steel surface and its environment. Paint retards the corrosion of the steel. Corrosion may manifest itself in many forms, and it may have many causes, but the effect is always the same: metal is consumed or deteriorated.

Paint, then, may be considered a low-cost renewable or repairable shield or membrane which is sacrificed to the elements to protect the metal. The service life expectancy of paint coats in California as affected by climatic conditions, is illustrated by the chart shown on Attachment No. 1 to this Bridge Construction-Memo.

The service life of a paint coat is also a function of the quality of the paint coat. Paint must be properly formulated and prepared from ingredients having certain necessary qualities. It must be

properly applied to clean surfaces of steel, and the completed film must have adequate thickness. Shortcomings in any of these requirements result in a decreased service life of the paint coat. In California, atmospheric conditions affecting the service life of paint coats vary between two extremes: the saline humidity of the sea coast and the hot aridity of the desert. Between these two extremes are regions where milder weather conditions prevail. Obviously, the need for protection is considerably less under mild exposures than it is under severe exposures. The paint system specified is therefore designed to meet the needs of the area, and conform to the latest pollution regulations imposed on solvent content of paint materials.

Current paint systems consist of either phenolic, or water-borne undercoats and water-borne finish coats. The thicknesses required varies according to the corrosion potential at the site.

Due to air pollution regulations, a paint system consisting of water-borne primers and top coats has been developed. This system consists of 4 mils of undercoat applied in 2 or more applications and 4 mils of finish paint applied in 2 or more applications. No vinyl wash primer is used.

Water-borne paints generally require higher temperatures and lower relative humidities than some other paints to dry properly. Care should be taken not to permit painting when the atmospheric or surface temperature is at or below 50 F, or when the relative humidity exceeds 75 percent. Temperature and relative humidity should remain within the above limits for approximately 4 hours after application to permit adequate drying.

From past experience with paint systems; the specification of multiple coat applications and the minimum dry film thicknesses of paint coats have evolved. Most paints used on structural steel. contain varying amounts of volatile solvents which, when they evaporate during the drying process, leave minute holes in the paint film. The application of multiple coats of paint, not too thin or too thick, tends to overcome the adverse pin-hole pattern in each coat and assures a truly impervious membrane. On any particular job, the specification of paint coat thickness of either paint system is adapted to the prevailing exposure conditions.

Most paints will not tolerate extra thick applications or puddles. If too much paint is applied, or puddles of the material are left on the surface, the coating will crack and lose bond with the steel or underlying coat. Each application should be held to near the amount specified.

Surface Preparation

The most important factor affecting the protective service life of a paint is the surface preparation prior to painting. The best paint available will not give optimum service when applied over improperly cleaned surfaces. It is essential, therefore, that paint is applied only to clean, sound, dry surfaces.

Although several methods of surface preparation are employed in the painting industry, it has been found that blast-cleaning and steam-cleaning are the most effective and least expensive methods. These two methods are specified almost exclusively. Occasionally, in mild exposure areas or where the type and amount of rust does not warrant the expense of blast-cleaning, hand cleaning methods may be specified.

Blast-cleaning is frequently referred to as sandblasting. However, since the abrasive used need not be limited to sand, the Office of Structure Construction has adopted the less restrictive term, blast-cleaning, in its specifications.

Blast-cleaning is simply the propulsion of an abrasive against an object, and the cleaning is accomplished by abrasive action. Various sources of power may be used to propel the abrasive, but the one most commonly used is compressed air. Another source is centrifugal force as used in large machines designed for the purpose. These machines are used only in shop installations because of their size and immobility. In field work, compressed air seems likely to remain the chief power source for some time. When dictated by adverse environmental impact, wet blast-cleaning may be specified. The power source for this method is either high pressure water or steam.

Sand, because of its abundance and consequent low cost, is the principal abrasive used. The only requirement imposed in the specifications is that the material be clean, dry, of proper grading, and meet requirements of the Air Resources Board for "Dry Unconfined Blasting". The degree of hardness is not specified.

Sand, obtained from commercial sources generally meets our requirements. Use of unwashed beach or river sand is not permitted because contaminants or too many fines are often present, It does not meet ARB requirements.

Other abrasives used on a lesser scale are steel shot, steel grit and slag from copper, nickel, and silver smelting processes. The use of steel shot or steel grit is usually limited to shop blasting where recovery for reuse is possible. High initial cost and lack of a practical recovery method prohibit the use of these abrasives in the field.

In the 1988 Standard Specifications "Blast-Cleaning," has been described in a different manner. This description is not intended to lower the degree of cleanliness of the steel from past years, but conforms more closely to language used in outside Industry.

Steam-cleaning consists of washing the surface to be painted with steam in which a biodegradable detergent soap has been incorporated in the feed water, or applied directly to the surface to be cleaned. The steam is directed against the surface, and the contaminants, loosened by the detergent, are carried away by the flushing action of the condensed steam. Any residue remaining on steam-cleaned surfaces should be flushed with fresh water before painting.

Steam-cleaning is used principally in maintenance work when spot-cleaning and painting are specified, although it may occasionally be used on shop-coated steel-in new construction work if the surfaces have become contaminated by dust, oil or other contaminating products. The primary purpose of steam-cleaning is to remove surface contaminants which would impair bonding of new paint to existing coatings. Steam-cleaning will not remove rust, and if rust is present after steam-cleaning, the operation will generally be followed by spot blast-cleaning.

An interval of at least 24 hours should elapse after steam cleaning before paint is applied.

A steam-cleaning supplement which describes the operation, equipment used and the detergent intermingling procedure in more detail is available upon request from the Office of Structure Construction.

Paint Application

The paint coats specified generally consist of one or more undercoats. The various coats or layers are planned and specified (1) to achieve an impervious membrane which inhibits corrosion: (2) to protect the steel against impact or abrasion and (3) to give the structure a pleasing appearance.

The normal functions of undercoats are to inhibit corrosion, to provide a suitable base for the finishing coats and to present a secondary barrier to any moisture penetrating the finishing coats.

Finishing coats comprise the tough outer layer of the paint film which is directly exposed to the weather. They are the weathering or wearing coats of a paint system and must, therefore, have a harder, more impervious surface than the undercoats. Two applications of finishing coat paint are normally specified.

Paint may be applied to structural steel by brush, roller or spray, but regardless of the method used, care must be exercised in the application in order that the maximum service-life may be realized. It is the responsibility of the Structure Representative to see that the paint is applied properly. The paint should be well mixed and uniformly applied, and any skips or holidays should be picked up before subsequent applications are allowed, since the smallest break or thin spot in the paint film is a potential trouble spot.

All formulations now in use, EXCEPT the inorganic zincs can be applied by any of the previously mentioned methods. Spraying is the only satisfactory method for application of inorganic zincs to large surfaces. However, small holidays or skips which sometimes occur around rivets or bolts can be picked up with a brush, and areas inaccessible with a spray gun should be swabbed or brushed.

Experience has taught us that "Airless" spray is inferior to conventional spray, on most bridge structures, due to lack of control of the amount of paint material being dispersed from the nozzle.

Paints for use on structural steel, except inorganic zinc primer, are manufactured ready for-application and thinning is not necessary, nor should it be tolerated. Inorganic zinc primer may be thinned as recommended by the manufacturer.

Painting for appearance may be considered of secondary importance to painting for protection, but it is evident that the public is aware of bridge appearance. Both maximum protection and pleasing appearance can be achieved by a paint job properly done. The most common causes of poor appearance are runs or sags in the paint film and paint spray or splatters on the concrete portion of the structure. By using care and precaution, it is far easier to prevent-these defects than it is to correct them.

Thickness of Paint Film

Dry film thickness of the paint film is always specified in either the special provisions or, by reference, in the Standard Specifications. In all cases, the specified mil thickness is the minimum on all surfaces and does not mean the overall averages.

Paint dry film thickness is measured by a magnetic flux gauge called "E1cometer" or "Positector" Gauges are supplied by the Office of Structure Construction with instructions for their use. These devices are delicate and expensive instruments and should, therefore, be handled with care. Gauges should not be stored near active electrical circuits, and they should not remain near welding equipment longer than absolutely necessary. Periodic checks to

determine the accuracy of the gauge is necessary: these checks may be made by using the shims provided. It is not the intention of the Office of Structure Construction to penalize a Contractor by requiring more thickness than specified, but, on the other hand, we should be sure that we do not get less. All measurements should be taken with the gauge placed firmly at right angles to the area being measured, even a slight slanting of the device gives a high reading, as will lack of solid contact. Recalibrate gauges on different types and sizes of steel. Reading differences have been noted between webs, stiffners and braces.

It is often necessary on small jobs or near the completion of large ones to measure a film thickness of paint which is not hard enough to prevent indentation by the film-thickness gauge. If a close inspection shows such a condition, the reading is certain to show less thickness than is actually on the steel. Correction can be made by placing a shim between the paint film and the film thickness, gauge and deducting the thickness shown on the shim from the reading taken.

The importance of adequate paint film thickness cannot be overstressed. All other things being equal, it is one of the factors that determines the service life of a paint job. It follows, therefore, that sufficient measurements should be taken to assure specified thicknesses in all places.

Protective Measures

Inherent in a bridge painting operation is the possibility of the creation of a nuisance or of the physical damage to adjacent property or to the traveling public. This is particularly true on contracts involving the repainting of structures under traffic.

Although the responsibility for the prevention of damage rests with the Contractor, the Structure Representative must constantly be aware of the job situation and should not hesitate to call the existence of hazards or potential sources of damage to the Contractor's attention.

In the event passing automobiles are spattered with paint, little damage will occur if the paint is immediately removed with mineral spirits or with water for water-borne paints. However, this should not be a common occurrence. A prudent Contractor will use drop cloths, screens, overhead tarps, and the like to adequately protect passing traffic or adjacent property.

Particular emphasis should be placed on the protection of concrete surfaces which are a part of the structure. The Contractor should not be allowed to mix paint or charge paint pots on bridge decks without adequate drop cloths. It is next to impossibleto remove paint-from concrete, and particular care should be exercised to prevent spattering such surfaces. Thinners and paint removers should not be used in attempting to remove paint from rough concrete surf aces. After the paint is dry, the area should be rubbed with a stone and wire brushed, or lightly blast-cleaned.

Paint which is being sprayed can drift as much as a quarter mile or more, and Contractors should be reminded of this possibility, particularly if automobiles are being parked nearby.

In general, the best protective measure is the anticipation of possible damage and prevention of its occurrence.

Paint Records and Reports

The Office of Structure Construction has developed a series of special record forms for use in keeping daily job records on each phase of the cleaning and painting operation. These forms will be furnished to the Structure Representative at the beginning of his assignment to a particular project, Samples of these forms (DH-OS M5, DH-OS M8, DH-OS M11, and DH-OS M78) are included in Section 16 of the Bridge Construction Records and Procedures.

Paint record sheets were developed to simplify the reporting of statistical data as well as to ensure uniformity in record keeping. Structure Representatives should be familiar with the use of the sheets and should enter the required information in accordance with procedures recommended by the painting section.

In addition to the paint records. The Resident Engineer's and/or Assistant Resident Engineer's Daily Reports (HC-1O and HC-1OA) are required for the painting operation.

The Blast-cleaning and paint record form (DH-OS M8, Daily clean and Paint Record), is a diary form used by the Structure Representative for the various phases of the cleaning and painting work. These diaries have the same significance as the general diary forms HC-10 and HC-1OA and should, therefore, receive the same degree of care in their preparation and distribution.

Form DH-OS M78, is a record of spot-blast-cleaning performed. The purpose of this form is to have the Structure Representative and the Contractor's representative agree, on a daily basis, on the amount of spot-blast-cleaning performed.

On repainting projects, the Structure Representative will prepare, from the information gathered in the daily diaries, cost data for the various phases of blast-cleaning and painting. This data will be entered on the paint data sheets, Form DH-OS M5, Clean and Paint Cost Summary. Use of this form aids the Structure Representative in making a systematic and uniform record of cost data.

Following completion of the painting operation, statistical information included on the paint record sheets is summarized on a special summary sheet, Form DH-OS M11, Paint Record. The primary purpose of the information summarized on this form is to provide a sound basis for estimating the cost of future painting projects.

It also provides information regarding the type and quantities of paint used, which information will be valuable when negotiation with pollution agencies in regards to removing paint and repainting the structure on future painting projects. Therefore, the form must be carefully and accurately completed if it is to have any real value.

The original Form DH-OS M11 and supporting paint record forms are retained in the job files until project completion, at which time they are submitted to the Sacramento Office with the Report of Completion. One copy of Form DH-OS M11 should be sent to the Office of Structure Construction as soon as the painting is completed.

Surface Area Computations

The area to be painted is an important part of the paint inspection procedure the surface area must be known to enable the Structure Representative to determine the true rate of progress and to calculate coverage rates. Surface area calculations are also of great value in the planning of future painting contracts. Surface areas of most structures are available in the Sacramento Office of Structures Maintenance. If they are not available, it will be the responsibility of the Structure Representative assigned to the project to calculate them. All calculations should be clearly shown so they may be easily checked by another person. Include subtotals for each span and separate summary sheets for each structure in the project.

Surface area computations will be submitted with Form DH-OS M11 as an attachment to the Report of Completion.

On request, charts to assist in the calculations of surface areas will be furnished to the project by the Office of Structure Maintenance.

Standard Paints Used by the Office of Structure Construction

The different types of paint currently being used by the Office of structure Construction are identified on Attachments No. 2 and 3 of this Bridge Construction Memo. Attachment No. 2 is a list of the standard paints used by the Office of structure Construction and Attachment No. 3 is a working list of approved water-borne inorganic zinc rich primers. No paint brand shall be used unless it is on the Departments current list of approved paints or meets the specifications of the Departments standard paints. In order to give a complete listing of allpaints which may possibly be specified in Office of Structure Construction work, we have included the specifications and descriptions of wood and concrete paints in the tabulation, although the painting of wood or concrete is not discussed in this memo.

Environmental Protection

It is the intention of the Office of Structure Construction to comply with regulations imposed by various public environmental protection agencies. These enforcement agencies are now operative in most areas of the state. Their primary concern is air and water pollution as well as noise abatement.

In order to comply with present and foreseen regulations, new cleaning and painting procedures have been and are being developed. In general, dust created by blast-cleaning, ground pollution from old lead and zinc paints, and overspray from paints are the chief offenders to the environment.

Curtailment methods for dust and waste products include confinement within the immediate work area and use of abrasives which create less dust. Wet-blast cleaning may be another alternative, subject to approval of the engineer.

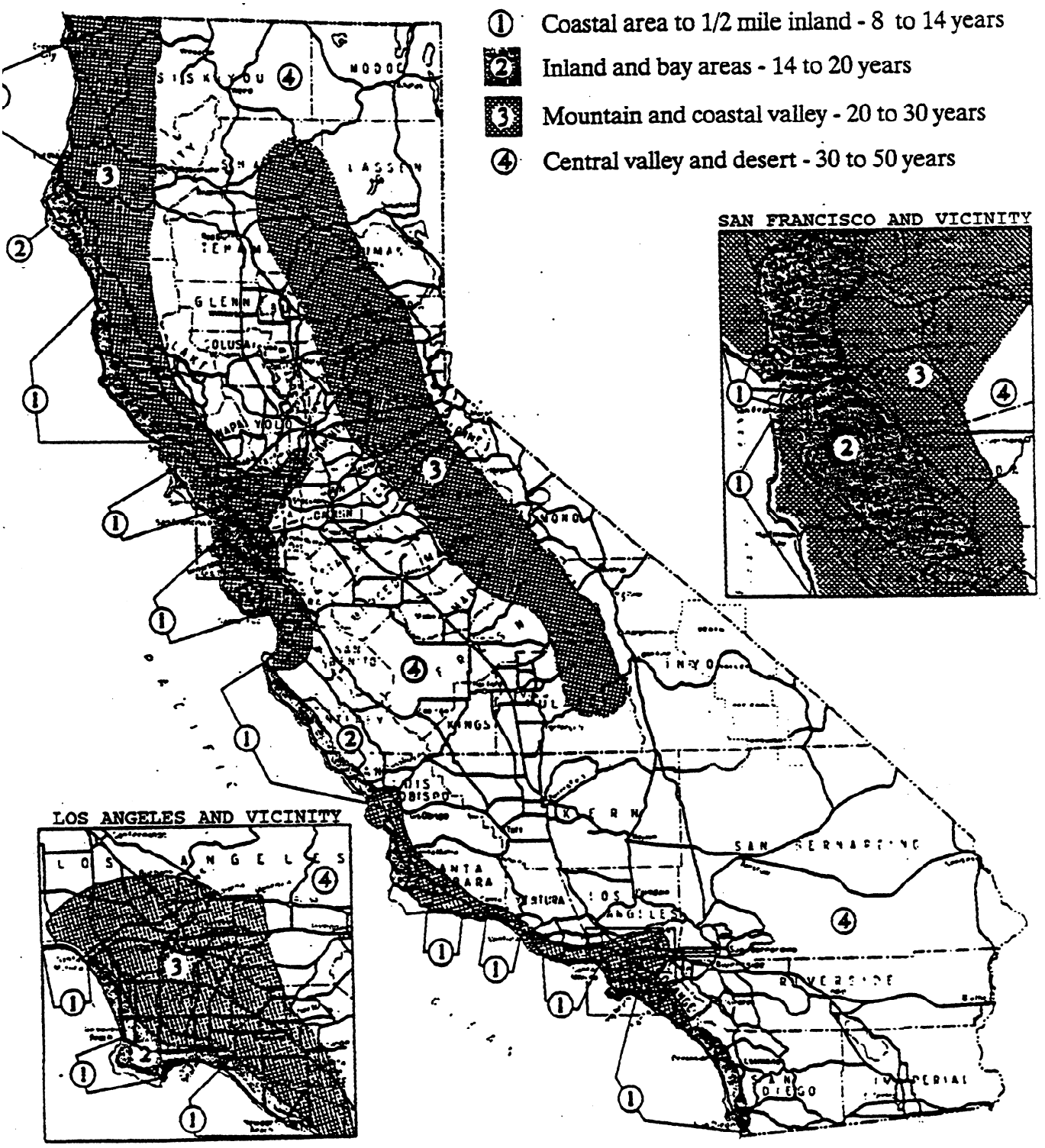
Confinement of waste products and dust is accomplished by using water curtains, planking, or by draping tarps, potato sacking, heavy-duty polyethylene bags or sheets, or similar materials around and under the work space. The confined waste materials are then collected and hauled to an approved dump site by an authorized transporter.

Copper, silver and nickel slags are sources of abrasives now in fairly common use. These abrasives are more expensive than sand. All abrasives, for dry, unconfined blasting including sand, must be approved by the Air Resources Board.

Wet-blast cleaning when it is specified may be done by two methods. One method is the so-called "ring" method. It consists of a perforated ring attached to the blasting nozzle. Water mist forced through holes in the ring mixes with the abrasive at the nozzle and effectively inhibits dust. However, rusting starts immediately on -freshly blasted metal, and the method is impractical for that reason. It is an effective method to use when blast-cleaning concrete, stucco or wood. The other method uses high pressure water or steam as the abrasive impellent. A dilute solution of sodium nitrite added to the water or steam inhibits rusting until the prime coat can be applied. The later method can be used, when necessary, provided excess water can be controlled. Both methods are more costly than dry blast-cleaning and will not be specified unless necessary to meet environmental regulations.

Lead pigmented paints are no longer being specified-for use on structural steel because of their toxicity.

PAINT SERVICE LIFE ON STRUCTURAL STEEL BRIDGES



Information shown is approximate only, compiled from records of the Division of Highways for existing bridges on the State Highway system.

BRIDGE CONSTRUCTION MEMO 155-1.0
ATTACHMENT No. 1 (Rev. 12-1-88)
Sheet 1 of 1

STANDARD PAINTS USED BY OFFICE OF STRUCTURE CONSTRUCTION

Phenolic Paints

Spec. No.	Name	Function	Used on
PB-201	Red Primer, High Solids Phenolic Type	Primer	Steel
PB-202	Pink Primer, High Solids Phenolic Type	Primer	Steel
PB-199	Aluminum Phenolic Tung Oil	Finish Paint	Steel

Water-Borne Paints

Spec. No.	Name	Function	Used on
PWB-142	Red Water-borne,	Primer	Steel
PWB-143	Pink Water-borne,	Primer	Steel
PWB-145	Red Water-borne,	Primer	Steel
PWB-146	Pink Water-borne,	Primer	Steel
PWB-87	Flat Gray Water-borne	Finish Paint	Steel
PWB-88	Light Tan Water-borne	Finish Paint	Steel
PWB-89	Tan Water-borne	Finish Paint	Steel
PWB-151	Aluminum, Leafing or Nonleafing Water-borne	Finish Paint	Steel
PWB-82	Light Green, Water-born	Weathering Coat	Steel
PWB-83	Green, Water-borne,	Weathering Coat	Steel
PWB-86	White Tintable, Water-borne	Weathering Coat	Steel
TT-P-19	Acrylic Emulsion - Tintable	Weathering Coat	Masonry
Fed. Spec. TT-P-001984	Wood Primer	Primer & Undercoat	Wood
Fed. Spec. TT-P-96D	White Wood Finish Coat	Weathering Coat	Wood

CALIFORNIA DEPARTMENT OF TRANSPORTATION

QUALIFIED PRODUCTS LIST

WATERBORNE INORGANIC ZINC RICH PRIMER

The following products have been evaluated and determined to provide a material meeting specification requirements for a waterborne inorganic zinc rich primer used in undercoating properly prepared structural steel in transportation maintenance and construction projects.

INORGANIC COATINGS INC.
IC 531 INORGANIC ZINC RICH PRIMER
(800) 345-0531

VALSPAR CORP.
MZ-6 HI-RATIO INORGANIC ZINC RICH PRIMER
(818) 334-8251

DEVOE COATINGS CO.
CATHACOTE 309 WATER BASED INORGANIC ZINC COATING
(504) 272-2470

DU PONT COATINGS CO.
GANICIN 347WB WATER BASED INORGANIC ZINC
(800) 346-4748

The effective period for this list is indeterminate. Other products will be considered for inclusion on this list subject to evaluation and approval by:

California Department of Transportation
Office of Transportation Materials and Research
5900 Folsom Boulevard
Sacramento, CA 95819

CHECK LIST FOR BRIDGE PAINTING PROJECTS

1. Check to see that steam cleaning is doing a satisfactory job of removing all dirt, grease, loose chalky paint, or other foreign materials.

2. Check to see that the specified biodegradable detergent is being used.

3. Check spot-blasting to be assured that all rust has been removed.

4. Check 100% blasted areas to be assured that all rust and old paint has been removed.

5. For "Spot Jobs", check to see that the air pressure and nozzle size meet the specifications.

6. Measure and record spot blast areas daily.

7. Check to see that the first coat of paint is being applied daily. If not, be sure that areas are reblasted before paint is applied.

8. Visually inspect backsides of rivets, tops of diaphragms, tops of bottom flanges, and other hard to reach areas to be assured that they are properly cleaned, and have the required paint coverage.

9. Check to be sure that the access to the work is adequate and that work areas are safe.

10. Observe mixing of paint materials to be assured that the mixing is being-properly done.

11. Require the Contractor to provide safe access to the work so that it can be properly inspected.

12. Check the temperature and humidity at intervals as required to be assured of specification compliance.

13. Check structural steel to be assured that it is dry when paint is applied.

14. Check undercoat for proper thickness before permitting the application of finish coats.

15. Record the quantities of abrasives and paint materials used daily. Also record man hours and hours of equipment use daily. This information is required for the Final Report

16. Enforce the specification requirements concerning the containment of fall-out materials.

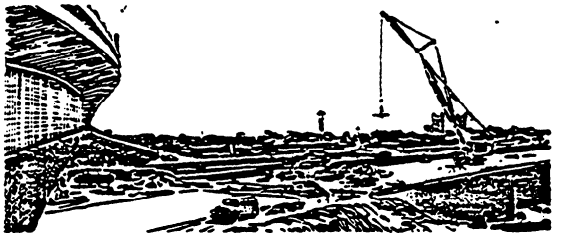
17. Enforce the specification requirements concerning the disposal of used sand and old paint.

18. Check to see that the Contractor is properly protecting deck soffit concrete, concrete caps, concrete piers and other concrete from overspray paint. Areas not so protected must be cleaned before the project is accepted.

19. Check-to see that the Contractor is taking proper precautions to prevent damage to adjacent trees, rocks, and property improvements.

20. Check to see that the Contractor is complying with the OSHA safety requirements.

21. Check to see that waste materials are collected and disposed of properly.



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BRIDGE PAINTING -- ESTIMATING WORK DONE

The attached summary sheet (Attachment #l) shows the percentage of the total work included in each phase of a typical bridge painting operation for paint systems used by the Office of Structure Construction.

Although the information shown is approximate only, it is sufficiently accurate for estimating purposes and may be used when computing amounts due on progress pay estimates for work performed under lump sum items.

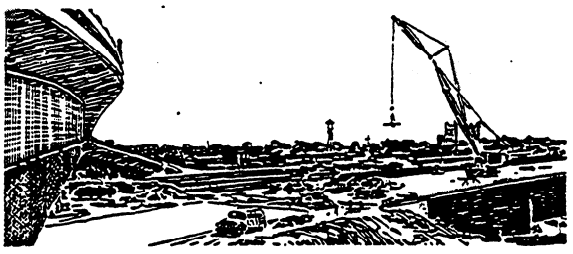
SUMMARY SHEET
PERCENT OF WORK IN LUMP SUM ITEMS

Shop Blast with Inorganic Zinc Water-borne Finish

Operation	Percent	Cumulative Percent
Blast Clean	40	40
Shop undercoats	30	70
Spot clean and undercoats in field	12	82
First finish coat	9	91
Final finish coat	9	100

100% Repaint with Water-borne Paint

Blast clean	60	60
First undercoat	10	70
Second undercoat	10	80
First finish coat	10	90
Second finish coat	10	100



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SAMPLING AND TESTING PAINT

The following sampling procedures are to be followed in order to be assured that all paint which is to be applied to permanent portions of the work complies with the Contract Specifications.

After the Contractor places an order for paint with a manufacturer, the manufacturer must notify the Transportation Laboratory of his intent to manufacture and package the paint. Transportation Laboratory personnel will then visit the manufacturing facility and take samples of the batch of paint. Depending on the manufacturer, and the type of paint, it may either be sampled before or after packaging. In either case the paint is identified by the manufacturers batch number and the date of manufacture. At the time of sampling, the State Inspector assigns a State lot number to the batch which then becomes a part of the identification.

When the testing is completed, the Transportation Laboratory notifies the manufacturer of the test results. If it is confirmed that the batch meets specifications, the manufacturer can then package and label the paint, or just label it if the paint had been packaged prior to taking the sample, The labeling should be in accordance with Section 91-1.03 of the Standard Specifications which states that "All containers of-paint shall be labeled showing the exact title of the paint specification, State specification number, manufacturer's name, date of manufacture, State lot number, and manufacturer's batch number". In addition to this, the State Inspector will place white inspection tags or stickers on some of the containers when he releases the paint to the jobsite.

These inspection release tags may or may not have the same State lot number as shown on the manufacturer's label. The lot number on the release tag should be checked against the Report of Inspection (R-29) which the Inspector should forward to the jobsite within a day or two of releasing the paint.

When the paint is delivered to the jobsite, the Structure Representative is to randomly select one container of each batch and ship it to the Sacramento Transportation Laboratory in its original unopened condition. Sample identification form DCR-TL-101 is to be filled in and sent in with the sample. No paint is to be

applied to the structure until the paint in this random sample has been tested by the Transportation Laboratory, and a test report has been issued confirming that the paint complies with the Contract Specifications. The test report is normally mailed to the jobsite; however, results of the testing may be obtained, by phone if necessary. The sample should show his telephone number on the sample identification form if he wishes to be notified of the test results by phone. Results can generally be obtained from the Transportation Laboratory within three to ten days after the Laboratory has received the sample.

One quart of paint is used in the paint testing procedure. The remainder of the paint in the container will be returned to the Contractor for use. The sample identification card should give the address to which the unused paint is to be returned. The unused paint cannot be returned to a P.O. Box Number.

The Standard Specifications permit the use of other than steel containers provided that the containers shall comply with U.S. Department of Transportation or the Interstate Commerce Commission regulations.

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 155-4**
	PAINT
Approved: FOR **R. P. SOMMARIVA, Chief** **Office of Structure Construction**	**Date: February 20, 1997** **Expires: March 1, 1998** **Supersedes: AJ96.7**

Subject: Application of Zinc-Rich Primer on Column Casings

It has come to our attention that some contractor's are using rollers to apply zinc-rich primer for repairs and touch up at welds on column casing jobs.

Section 59-2.13, "Application of Zinc-Rich Primer", of the Standard Specifications requires zinc-rich primer to be applied by the spray method and that an agitating pot be used.

Section 59-2.13, of the Standard Specifications, does allow zinc-rich primer to be applied by brush, dauber or roller if the area to be painted is inaccessible to spray application. Column casings are not to be considered inaccessible for spray applications.

On going column casing contracts, any zinc-rich primer not yet top coated and applied by any means other than spray methods, should be removed and reapplied using spray methods in accordance with Section 59-2.13. No additional compensation should be allowed for compliance with Section 59-2.13.

cc: Str. Reps
BCEs
ACMs
Consultant Firms
BGauger, Construction Program Manager

BRIDGE CONSTRUCTION MEMO 160-0.0

PRESTRESSED CONCRETE

January 15, 2005

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DOLORES M. VALLS, Deputy Division Chief
Offices of Structure Construction

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Volume II

BROKEN OR SLIPPED PRESTRESS STRANDS

Structure Construction's policy when prestressing strands break and it has been determined by the Engineer that it is structurally satisfactory to leave the broken strands out, is to give the contractor the option to replace the broken strands or request a contract change order with a rebate to the State.

The rebate should be arrived at as follows:

$$\text{Rebate} = \frac{\text{Total Lineal feet broken strand}}{\text{Total lineal feet strand on the job}} \times \text{Contract item price for prestressing}$$

In the case of slipped strand, when it has been determined that it is not feasible to re-grip and stress to the required force and all the other acceptance criteria are met, the same procedure may be followed.

Volume II

PATCHING CONCRETE UNDER PRESTRESS BEARING PLATES

Epoxy tends to creep or flow under sustained high stress. Therefore, when patching or replacing concrete immediately behind a prestress bearing plate or in the bearing seat area, the repair shall not be made with material that uses epoxy as a binder. However, epoxy may be used for bonding the repair material to the existing concrete.

Extensive repairs should be re-poured rather than dry-packed. The concrete used in the repair area should have attained the strength required for the structure concrete before the stressing operation is started.

Volume II

PRESSURE CELLS

Structure construction utilizes electro-hydraulic pressure cells combined with a strain indicator (read-out box) to perform QA verification of the contractor's mechanical pressure gage(s) during the post-tensioning operation. Many refer to this equipment incorrectly as a "load cell".

The differences between load cells and pressure cells are discussed below.

A **load cell** converts an applied force into a proportional voltage change. Inside the load cell is a **force** transducer. The force transducer contains internal strain gages. These strain gages consist of fine wire elements that, when stretched, change electrical resistance. Four strain gages are arranged inside the force transducer to form a "Wheatstone Bridge Circuit". This circuit allows for the precise measurement of input and output voltage across the loaded element. The strain gage indicator (read-out box) is used to measure the voltage differential across the transducer circuit. The voltage differential is converted to a direct load reading on the display of the strain gage indicator.

A **pressure cell** uses a **hydraulic pressure transducer** to convert an applied pressure into a proportional voltage change. Strain gages are used inside the pressure transducer in the same way as a force transducer. The strain gage indicator (read out box) is used to measure the voltage differential across the transducer circuit. The voltage differential is converted to a direct load reading on the display of the strain gage indicator.

Structure construction uses pressure cells, not load cells when monitoring post–tensioning operations. However, when METS calibrates the contractor's jacks and gages, they use both a load cell and a pressure cell.

Pressure cell units have been assigned to Senior Bridge Construction Engineers throughout the State. The OSC equipment database, available on the OSC intranet site, can be utilized to locate additional equipment, if needed. If further assistance is required obtaining a pressure cell unit or if repairs are needed, contact the OSC equipment manager at 916-227-7777.

Structure Representatives should make advance arrangements with their Construction Engineer to obtain a pressure cell unit. Senior Bridge Engineers will arrange for their personnel in the area to become proficient in the use of the pressure cell units.

Additional information is given in the current addition of the California Prestress Manual.

Volume II

STRESSING INCOMPLETE BRIDGES

On rare occasions, usually due to unforeseen emergency situations, contractors may desire to post-tension partially completed bridges. All requests to stress partially completed bridges should be discussed with the Bridge Construction Senior, the Area Construction Manager, and the Project Engineer.

The Office of Structure Design, Memo to Designers 11-18, outlines guidelines and will assist in providing statewide uniformity in responding to requests related to stressing of partially stressed bridges. A copy of Memo to Designers 11-18 can be found at: http://www.dot.ca.gov/hq/esc/techpubs/

Volume II

ELECTRIC WELDING OF PRESTRESS STRAND

Section 50-1.05 of the Standard Specifications prohibits electrical welding of prestressing strands after fabrication. Arc welding of the strand is an unsound practice and under normal circumstances shall not be allowed.

The issue of field welding prestressing strand typically arises when contractors want to arc weld the ends of the strands together to a pulling head. Welding the ends together prevents the individual strands from slipping in the bundle when the tendon is pulled through the duct.

The main concern with electrical welding of prestress strands is that stray current from the welding procedure may arc and pit a portion of the strand far from the actual weld location. The pitting damage to the strand may adversely affect the service life and performance of the post-tensioning system.

The use of pulling grips or non-electrical based welding of the strand ends (e.g. oxyacetylene brazing, chemical, etc) are acceptable methods for pulling long presstress strands. Pulling grips are used extensively in the electrical industry and have been successfully used to pull long prestress tendons. Regardless of the method used, all damage to the strands caused by the pulling system must be removed (cut back) from the portion to be incorporated into the final work.

As emphasized above, electrical welding of presstress strands is prohibited by the Standard Specifications and is generally understood to be unsound practice. However, under certain unique situations[1], arc welding a pulling head to the strands to facilitate strand installation may be the alternative with the least detrimental affects. A project specific exception to allow arc welding may be given if approved by the State Bridge Engineer and Deputy Division Chief of Structure Construction.

[1.] An exception was recommended and approved for the new Benicia-Martinez Bridge. This extremely complex structure had significantly long (approximately 1000 feet) and sharp radii tendon paths.

Volume II

PRESTRESSED CONCRETE WORKING DRAWINGS

Introduction

Structure Construction's procedure for review and approval of working drawings for prestressed concrete is a coordinated effort between Design and Construction personnel. Primary responsibility for approval of the working drawings rests with the Designer. On externally financed projects that are not State designed, primary responsibility for approval of the working drawings rests with the Local Agency Engineer or the Consultant Designer. The Liaison Engineer will have checked the drawings only in relation to the approval or disapproval of the prestress system and a Technical Specialist in Structures Design will have performed a concurrent, cursory review to ensure it is an approved system.

Working Drawings

Memo to Designers 11-1 "Review of Working Drawings, Prestressed Concrete" covers the procedures required for review and approval of working drawings, including responsibilities of Structure Representatives on construction projects. This memo can be found at the following link:

http://www.dot.ca.gov/hq/esc/techpubs/manual/bridgemanuals/bridge-memo-to-designer/bmd.htm - sec11

Normal procedure is for the Contractor (subcontractor or fabricator) to submit all working drawings directly to the Office of Structure Design, Documents Unit, Mail Station 9-4/4I, 1801 30th Street, Sacramento 95816. The Structure Representative is not to accept submittals.

The Documents Unit Group, Office of Structure Design, administers the working drawing approval procedure for all State jobs, including those that are externally financed. The group maintains a record of all working drawings submitted, and distributes copies to all interested parties, during all phases of the approval procedure. This relieves the Structure Representative of tedious administrative details necessary to insure that working drawings are distributed to the right people at the right time.

The responsibility for checking working drawings is shared by the Designer and the Structure Representative. Working drawings shall not be returned to the Contractor until the Designer has discussed and resolved the details with the Structure Representative. The comments returned to the Contractor must be acceptable to both the Designer and the Structure Representative.

Instructions to Structure Representatives

1. Comply with applicable instructions in Design Memo 11-1. Communicate directly with the Design Branch Chief, Designer, Liaison Engineer, or Design Consultant when necessary.

2. If the Contractor submits final working drawings to the Resident Engineer or Structure Representative, after reviewing the drawings to be sure that all field changes and minor corrections are noted, transmit all sets promptly to the Office of Structure Design, Documents Unit, Mail Station 9-4/4I, 1801 30th Street, Sacramento 95816.

3. Keep the Resident Engineer informed on the status of the final working drawing submittal so that the contract will not be finalized prior to fulfillment of all contract requirements. Working drawing status can be checked at 'Tracker Web' on the OSC website under the 'Field Resources' tab at the following address: http://onramp.dot.ca.gov/hq/oscnet/.

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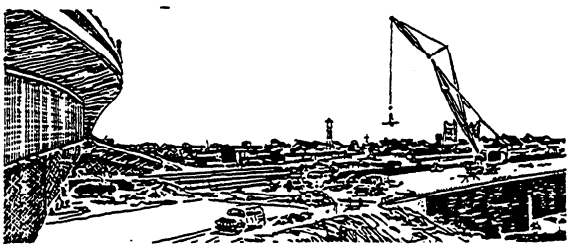

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161-1.0	4-27-87	PUMPING PLANT ELECTRICAL AND MECHANICAL EQUIPMENT MATERIALS LISTS AND WORKING DRAWINGS
161-2.0	12-04-81	ELECTRICAL SERVICE FOR PUMPING PLANTS

A. P. BEZZONE, Chief
Office of Structure Construction

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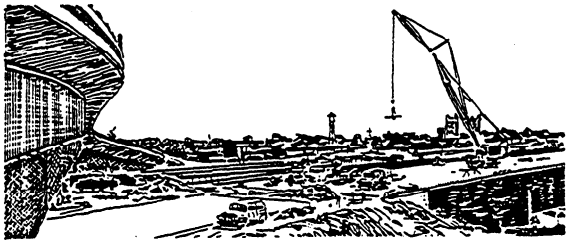

Volume II

PUMPING PLANT ELECTRICAL AND MECHANICAL EQUIPMENT MATERIALS LISTS AND WORKING DRAWINGS

Materials lists and working drawings for electrical and mechanical equipment, as required by Section 74-1.04 of the Standard Specifications, shall be checked and approved by the Mechanical and Electrical Section.

Normal procedure is for the Contractor (subcontractor or fabricator) to submit all working drawings directly to the Office of Structure Design, Document Unit, P. 0. Box 942874, Sacramento 94274-0001. The Structure Representative is not to accept submittals.

Near the beginning of contracts that have pumping plant electrical and/or mechanical work involved, a representative of the Mechanical and Electrical Section will consult with the Structure Representative and give any necessary instructions at that time.

ELECTRICAL SERVICE FOR PUMPING PLANTS

The contract "Special Provisions" specify the requirements for furnishing electrical service to pumping plants and also specify the provisions for handling the electrical service charges. Generally, the service charges become the obligation of the State upon acceptance of the contract, but may become the State's obligation at some other time during the life of the contract if so specified in the "Special Provisions".

The Structure Representative must notify the District Resident Engineer, so that he may notify the District Office sufficiently in advance to allow time for making arrangements with the utility company for continuing, changing, or discontinuing the service.

Volume II

BRIDGE CONSTRUCTION MEMO 162-0.0

RAILINGS AND BARRIERS

July 1, 1999

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Memo No.	Date	Title
162-1.0		BLANK
162-2.0	07-01-99	CONCRETE BARRIERS ON STRUCTURES

RALPH P. SOMMARIVA, Chief
Office of Structure Construction

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Volume II

BRIDGE CONSTRUCTION MEMO 162-2.0

RAILINGS AND BARRIERS

July 1, 1999

CONCRETE BARRIERS ON STRUCTURES

Concrete barriers Type 732 and 736 having constant sloping faces (1:6.25) on the traffic side are approved for use at the edge of deck of structures and retaining walls. Other approved safety shape concrete barriers for use on structures and retaining walls are Types 25, 27 and 28.

The Engineering Service Center has approved the use of the “experimental” Type 60A and Type 60GA for median applications on new structures only.

The type 60A and 60GA are heavier than the Type 50 and existing bridge decks may not carry the additional load. Any proposal to use the type 60A or 60GA on existing bridge decks should be discussed with your designer or design oversight engineer.

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 162-3 RAILINGS AND BARRIERS**
Approved:_____ **R. P. SOMMARIVA, Chief** **Office of Structure Construction**	**Date: October 1, 1998** **Expires: November 1, 1999** **Supersedes: None**

Subject: Finishing of Concrete Barriers

The final surface finish of concrete barriers shall conform to the provisions of section 83-2.02(D), Finishing, of the Standard Specifications. The specification allows the contractor to propose an alternative finishing method.

An allowable alternative is to provide a final surface finish utilizing non-abrasive methods such as a wet sponge finish with cementitious materials. The final surface must be smooth, with an even surface of uniform texture and appearance, free of unsightly bulges and other surface imperfections. The method is subject to approval by the Engineer.

c: BCR&P Manual Holders
Consultant Firms
BFelker, Construction Program Manager

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MEMO NO.	ISSUE DATE	TITLE
165-1.0	02/13/2012	Reinforcing Steel Hook Details
165-2.0	02/14/2007	Identification of Reinforcing Steel Bars
165-3.0	02/13/2012	Reinforcing Steel Bar Chart
165-4.0	02/13/2012	Welded Wire Fabric
165-5.0	02/13/2012	(Blank)
165-6.0	02/13/2012	(Blank)
165-7.0	10/31/2012	Reinforcement Splices
165-10.0	10/31/2012	(Blank)
165-11.0	04/29/2011	Allowing the Use of Plastic Spacers in Cast-In-Place Concrete Piles

Robert A. Stott, Deputy Division Chief
Division of Engineering Services
Structure Construction

**Denotes the document is a Bridge Construction Bulletin*

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Reinforcing Steel Hook Details

The Standard Specifications require that reinforcing steel hooks and bends conform to the provisions of the Building Code Requirements for Structural Concrete published by the American Concrete Institute (ACI).

The attached chart is from the Concrete Reinforcing Steel Institute (CRSI) Manual of Standard Practice, and conforms to the ACI 318 code requirements for standard hook details. The chart is provided to assist field personnel with the inspection of reinforcing steel.

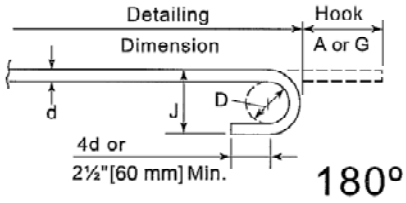
RECOMMENDED INDUSTRY PRACTICE FOR DETAILING REINFORCING MATERIALS

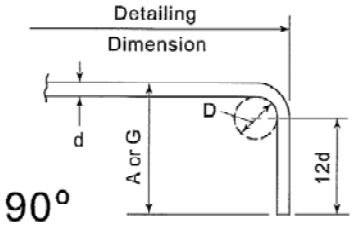
STANDARD HOOKS

All specific dimensions recommended by CRSI below meet minimum requirements of ACI 318 [318M].

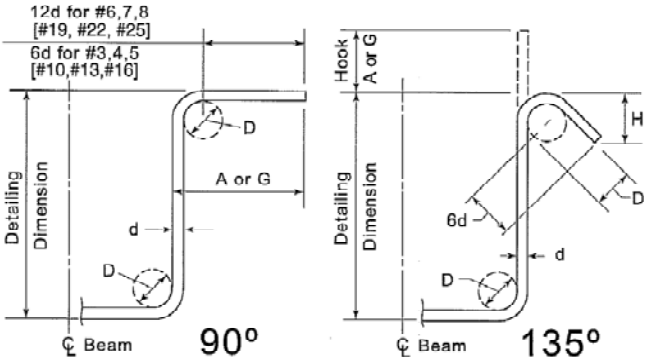
RECOMMENDED END HOOKS
All Grades of Steel
D = Finished bend diameter

Bar Size	D, in. [mm]	180° Hooks, ft-in. [mm]		90° Hooks, ft-in. [mm]
		A or G	J	A or G
# 3 [#10]	2¼ [60]	0-5 [125]	0-3 [80]	0-6 [150]
# 4 [#13]	3 [80]	0-6 [150]	0-4 [105]	0-8 [200]
# 5 [#16]	3¾ [95]	0-7 [175]	0-5 [130]	0-10 [250]
# 6 [#19]	4½ [115]	0-8 [200]	0-6 [155]	1-0 [300]
# 7 [#22]	5¼ [135]	0-10 [250]	0-7 [180]	1-2 [375]
# 8 [#25]	6 [155]	0-11 [275]	0-8 [205]	1-4 [425]
# 9 [#29]	9½ [240]	1-3 [375]	0-11¾ [300]	1-7 [475]
#10 [#32]	10¾ [275]	1-5 [425]	1-1¼ [335]	1-10 [550]
#11 [#36]	12 [305]	1-7 [475]	1-2¾ [375]	2-0 [600]
#14 [#43]	18¼ [465]	2-3 [675]	1-9¾ [550]	2-7 [775]
#18 [#57]	24 [610]	3-0 [925]	2-4½ [725]	3-5 [1050]

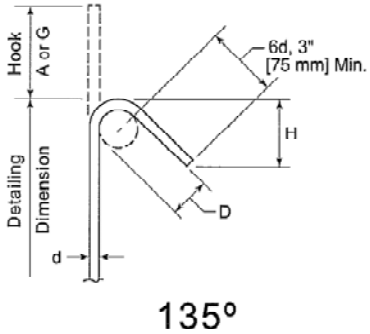




90° AND 135° STIRRUP AND TIE HOOKS



135° SEISMIC STIRRUP/TIE HOOKS



STIRRUP
(TIES SIMILAR)
STIRRUP AND TIE HOOK DIMENSIONS
ALL GRADES OF STEEL

Bar Size	D, in. [mm]	90° Hook, ft-in. [mm]	135° Hook, in. [mm]	
		Hook A or G	Hook A or G	H (Approx.)
#3 [#10]	1½ [40]	0-4 [105]	4 [105]	2½ [65]
#4 [#13]	2 [50]	0-4½ [115]	4½ [115]	3 [80]
#5 [#16]	2½ [65]	0-6 [155]	5½ [140]	3¾ [95]
#6 [#19]	4½ [115]	1-0 [305]	8 [205]	4½ [115]
#7 [#22]	5¼ [135]	1-2 [355]	9 [230]	5¼ [135]
#8 [#25]	6 [155]	1-4 [410]	10½ [270]	6 [155]

135° SEISMIC STIRRUP/TIE
HOOK DIMENSIONS
ALL GRADES OF STEEL

Bar Size	D, in. [mm]	135° Hook, in. [mm]	
		Hook A or G	H (Approx.)
#3 [#10]	1½ [40]	4¼ [110]	3 [80]
#4 [#13]	2 [50]	4½ [115]	3 [80]
#5 [#16]	2½ [65]	5½ [140]	3¾ [95]
#6 [#19]	4½ [115]	8 [205]	4½ [115]
#7 [#22]	5¼ [135]	9 [230]	5¼ [135]
#8 [#25]	6 [155]	10½ [270]	6 [155]

BRIDGE CONSTRUCTION MEMO 165-2.0

REINFORCING STEEL

February, 14 2007

Volume II

IDENTIFICATION OF REINFORCING STEEL BARS

ASTM specifications for reinforcing steel require identification marks to be rolled into the surface of the bars.

The attached pages are from Chapter 1 and Appendix A of the latest Manual of Standard Practice of the Concrete Reinforcing Steel Institute (May 2003). These reproduced pages illustrate the mill identification marks and other symbols designating size and grade of the reinforcing steel.

1.7. Identification Marks*—ASTM Standard Reinforcing Bars

The ASTM specifications for reinforcing bars require identification marks to be rolled into the surface on one side of the bar to denote the Producer's mill designation, bar size, type of steel, and minimum yield designation. Grade 60 [420] bars show these marks in the following order:

1st—Producing Mill (usually a letter)
2nd—Bar Size Number (#3 through #11, #14, #18 [#10 through #57])
3rd—Type of Steel:

S for Billet-Steel (A615/A615M)

W for Low-Alloy Steel (A706/A706M)

I for Rail-Steel (A996/A996M)

R for Rail-Steel (A996/A996M)

A for Axle-Steel (A996/A996M)

4th—Minimum Yield Strength Designation

A mark for minimum yield designation or grade is required for Grade 60 [420] and Grade 75 [520] bars only. Grade 60 [420] bars can either have one single longitudinal line (a grade line) or the number 60 [4] (a grade mark). Grade 75 [520] bars can either have two grade lines or the grade mark 75 [5].

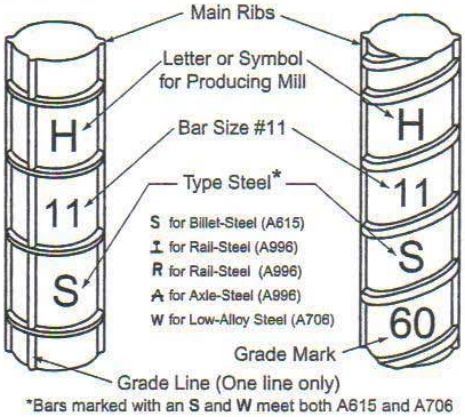
A grade line is smaller and is located between the two main longitudinal ribs which are on opposite sides of all bars rolled in the United States. A grade line must be continued through at least 5 deformation spaces, and it may be placed on the same side of the bar as the other markings or on the opposite side.

Grade 40 [300] and 50 [350] bars are required to have only the first three identification marks. No grade mark or grade line for minimum yield strength is required.

VARIATIONS: Bar identification marks may also be oriented to read horizontally (at 90° to those illustrated). Grade mark numbers may be placed within separate consecutive deformation spaces to read vertically or horizontally.

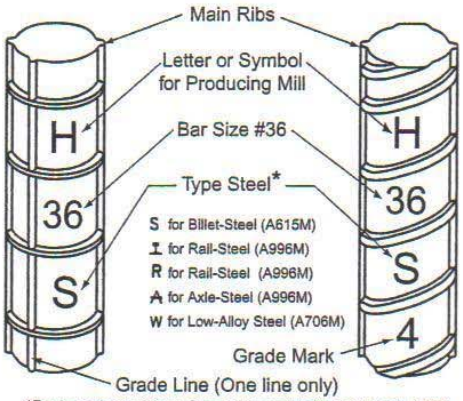
*See Appendix A for complete identification marks of Grade 60 [420] reinforcing bars produced by all U.S. Producers. The marks, listed alphabetically by producing mill, include the identification requirements of ASTM and the deformation pattern used by each mill.

MATERIAL SPECIFICATIONS FOR REINFORCING BARS



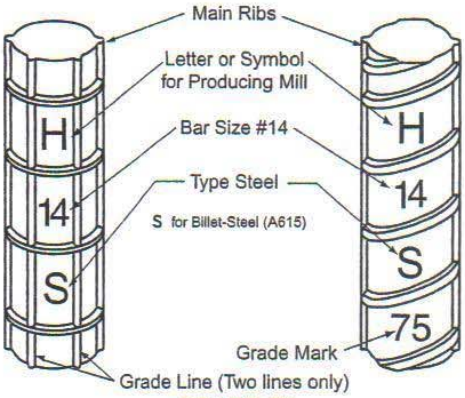
*Bars marked with an S and W meet both A615 and A706

GRADE 60

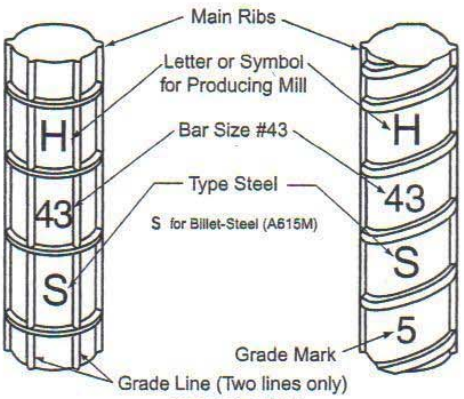


*Bars marked with an S and W meet both A615 and A706

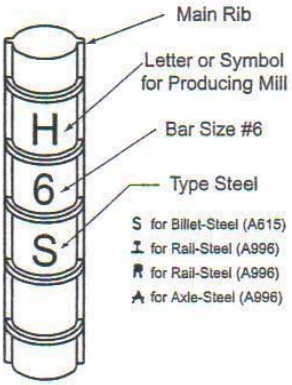
GRADE 420



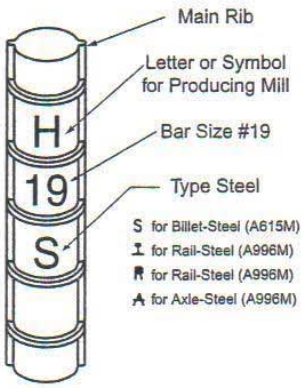
GRADE 75



GRADE 520



GRADES 40 AND 50



GRADES 300 AND 350

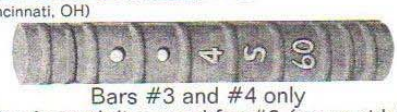
APPENDIX A

U.S. MANUFACTURERS OF GRADE 60 [420] CONCRETE REINFORCING BARS

ASTM and AASHTO Specifications require that all reinforcing bars be identified by permanent, mill imprinted markings.

1 A.B. STEEL MILL, INC.
(Cincinnati, OH)

S



Bars #3 and #4 only
Grade mark line used for #3 (same side)

1 A.B. STEEL MILL, INC.
(Cincinnati, OH)

S

Bars #5 through #8 only

2 AMERISTEEL CRSI
(Charlotte Steel Mill Division in Charlotte, NC)

S

Bars #13 through #25 only

2 AMERISTEEL CRSI
(Charlotte Steel Mill Division in Charlotte, NC)

W

Bars #13 through #25 only

2 AMERISTEEL CRSI
(Jacksonville Steel Mill Division in Baldwin, FL)

S

Bars #10 through #36 only (#10 through #16 coiled)

2 AMERISTEEL CRSI
(Jacksonville Steel Mill Division in Baldwin, FL)

W

Bars #10 through #36 only (#10 through #16 coiled)

2 AMERISTEEL CRSI
(Knoxville Steel Mill Division in Knoxville, TN)

S

Bars #13 through #36 only

2 AMERISTEEL CRSI
(Knoxville Steel Mill Division in Knoxville, TN)

W

Bars #13 through #36 only

2 AMERISTEEL CRSI
(West Tennessee Steel Mill Division in Jackson, TN)

S

Bars #10 through #36 only

2 AMERISTEEL CRSI
(West Tennessee Steel Mill Division in Jackson, TN)

S

Bars #43 and #57 only

2 AMERISTEEL CRSI
(West Tennessee Steel Mill Division in Jackson, TN)

W

Bars #10 through #36 only

2 AMERISTEEL CRSI
(West Tennessee Steel Mill Division in Jackson, TN)

W

Bars #43 and #57 only

3 AUBURN STEEL COMPANY, INC. CRSI
(Auburn Division in Auburn, NY)

S

Bars #13 through #43 only

3 AUBURN STEEL COMPANY, INC. CRSI
(Auburn Division in Auburn, NY)

W

Bars #13 through #43 only

3 AUBURN STEEL COMPANY, INC. CRSI
(Lemont Division in Lemont, IL)

S

Bars #13 through #36 only

3 AUBURN STEEL COMPANY, INC. CRSI
(Lemont Division in Lemont, IL)

W

Bars #13 through #36 only

4 BAYOU STEEL CORP.
(Harriman, TN)

S

Bars #10 through #36 only

5 BIRMINGHAM STEEL CORP. CRSI
(Alabama Steel Division in Birmingham, AL)

S

Bars #10 through #36 only

Note: CRSI mill members are in boldface with the CRSI logo at the top right corner.

U.S. MANUFACTURERS OF GRADE 60 [420] CONCRETE REINFORCING BARS

ASTM and AASHTO Specifications require that all reinforcing bars be identified by permanent, mill imprinted markings.

5 BIRMINGHAM STEEL CORP. CRSI
(Alabama Steel Division in Birmingham, AL)

W

Bars #10 through #36 only

5 BIRMINGHAM STEEL CORP. CRSI
(Illinois Steel Division, Joliet Rolling Mill in Joliet, IL)

S

Bars #10 through #19 only (#10 through #19 coiled)

5 BIRMINGHAM STEEL CORP. CRSI
(Illinois Steel Division, Joliet Rolling Mill in Joliet, IL)

W

Bars #10 through #19 only (#10 through #19 coiled)

5 BIRMINGHAM STEEL CORP. CRSI
(Illinois Steel Division, Kankakee Plant in Bourbonnais, IL)

S

Bars #10 through #36 only

5 BIRMINGHAM STEEL CORP. CRSI
(Illinois Steel Division, Kankakee Plant in Bourbonnais, IL)

W

Bars #10 through #36 only

5 BIRMINGHAM STEEL CORP. CRSI
(Mississippi Steel Division in Jackson, MS)

S

Bars #13 through #36 only

5 BIRMINGHAM STEEL CORP. CRSI
(Mississippi Steel Division in Jackson, MS)

W

Bars #13 through #36 only

5 BIRMINGHAM STEEL CORP. CRSI
(Seattle Washington Steel Division in Seattle, WA)

S

All bar sizes

5 BIRMINGHAM STEEL CORP. CRSI
(Seattle Washington Steel Division in Seattle, WA)

W

All bar sizes

6 BORDER STEEL, INC. CRSI
(El Paso, TX)

S

Bars #10 through #36 only

6 BORDER STEEL, INC. CRSI
(El Paso, TX)

S

Bars #43 and #57 only

6 BORDER STEEL, INC. CRSI
(El Paso, TX)

W

Bars #10 through #36 only

6 BORDER STEEL, INC. CRSI
(El Paso, TX)

W

Bars #43 and #57 only

7 CASCADE STEEL ROLLING MILLS, INC. CRSI
(McMinnville, OR)

S

Bars #10 through #19 only (#10 through #19 coiled)

7 CASCADE STEEL ROLLING MILLS, INC. CRSI
(McMinnville, OR)

S

Bars #16 through #22 only

7 CASCADE STEEL ROLLING MILLS, INC. CRSI
(McMinnville, OR)

S

Bars #25 through #57 only

7 CASCADE STEEL ROLLING MILLS, INC. CRSI
(McMinnville, OR)

W

Bars #13 through #19 only (#13 through #19 coiled)

7 CASCADE STEEL ROLLING MILLS, INC. CRSI
(McMinnville, OR)

W

Bars #16 through #22 only

Note: CRSI mill members are in boldface with the CRSI logo at the top right corner.

U.S. MANUFACTURERS OF GRADE 60 [420] CONCRETE REINFORCING BARS

ASTM and AASHTO Specifications require that all reinforcing bars be identified by permanent, mill imprinted markings.

7 CASCADE STEEL ROLLING MILLS, INC. (McMinnville, OR) **CRSI**

W

Bars #25 through #57 only

8 CHICAGO HEIGHTS STEEL (Chicago Heights, IL)

I

Bars #4 through #8 only

9 CONNECTICUT STEEL CORP. (Wallingford, CT)

S

Coiled bars #10 and #13 only

10 CO-STEEL RARITAN (Perth Amboy, NJ)

S

Coiled bars #10 through #19 only

10 CO-STEEL RARITAN (Perth Amboy, NJ)

W

Coiled bars #10 through #19 only

11 CO-STEEL SAYREVILLE (Sayreville, NJ)

S

All bar sizes

11 CO-STEEL SAYREVILLE (Sayreville, NJ)

W

All bar sizes

12 GST STEEL COMPANY (Kansas City, MO)

S

Coiled bars #3 and #4 only

12 GST STEEL COMPANY (Kansas City, MO)

W

Coiled bars #3 and #4 only

13 MARION STEEL COMPANY (Marion, OH) **CRSI**

S

Bars #13 through #36 only

Grade mark line opposite side,"I" may be under "S" for #13 or #16

13 MARION STEEL COMPANY (Marion, OH) **CRSI**

W

Bars #13 through #36 only

Grade mark line on opposite side

14 NORTH STAR STEEL COMPANY (Beaumont Mill in Beaumont, TX) **CRSI**

S

Coiled bars #10 through #16 only

14 NORTH STAR STEEL COMPANY (Kingman Mill in Kingman, AZ) **CRSI**

S

All bar sizes (#10 through #16 coiled)

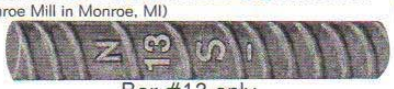
14 NORTH STAR STEEL COMPANY (Kingman Mill in Kingman, AZ) **CRSI**

W

Bars #10 through #36 only (#10 through #16 coiled)

14 NORTH STAR STEEL COMPANY (Monroe Mill in Monroe, MI) **CRSI**

S



Bar #13 only

Grade mark line on opposite side

14 NORTH STAR STEEL COMPANY (Monroe Mill in Monroe, MI) **CRSI**

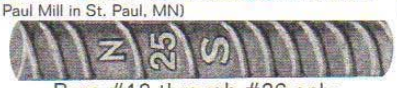
W

Bar #13 only

Grade mark line on opposite side

14 NORTH STAR STEEL COMPANY (St. Paul Mill in St. Paul, MN) **CRSI**

S

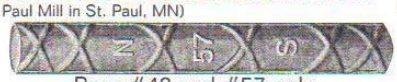


Bars #13 through #36 only

Grade mark line on opposite side

14 NORTH STAR STEEL COMPANY (St. Paul Mill in St. Paul, MN) **CRSI**

S



Bars #43 and #57 only

Grade mark line on opposite side

Note: CRSI mill members are in boldface with the CRSI logo at the top right corner.

U.S. MANUFACTURERS OF GRADE 60 [420] CONCRETE REINFORCING BARS

ASTM and AASHTO Specifications require that all reinforcing bars be identified by permanent, mill imprinted markings.

14 NORTH STAR STEEL COMPANY **CRSI**
(St. Paul Mill in St. Paul, MN)

W

Bars #13 through #36 only
Grade mark line on opposite side

14 NORTH STAR STEEL COMPANY **CRSI**
(St. Paul Mill in St. Paul, MN)

W

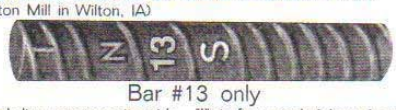
Bars #43 and #57 only
Grade mark line on opposite side

14 NORTH STAR STEEL COMPANY **CRSI**
(St. Paul Mill in St. Paul, MN)

S

Bars #19 through #57 (Patented)
Long. groove one side only, marking system not per ASTM

14 NORTH STAR STEEL COMPANY **CRSI**
(Wilton Mill in Wilton, IA)

S 

Bar #13 only
Grade mark line on opposite side - "I" is first mark (shown) or under S

15 NORTHWESTERN STEEL & WIRE CO.
(Sterling, IL)

S

Coiled bars #10 through #16 only
Inch-pound markings on opposite side

15 NORTHWESTERN STEEL & WIRE CO.
(Sterling, IL)

S

Coiled bars #10 through #16 only
Inch-pound markings on opposite side

16 NUCOR STEEL **CRSI**
(South Carolina Mill in Darlington, SC)

S

Bars #13 through #19 only (#16 coiled)

16 NUCOR STEEL **CRSI**
(South Carolina Mill in Darlington, SC)

S

Bars #22 through #36 only

16 NUCOR STEEL **CRSI**
(Texas Mill in Jewett, TX)

S

Bars #10 through #36 only

16 NUCOR STEEL **CRSI**
(Utah Mill in Plymouth, UT)

S

Bars #10 through #25 only (#10 through #16 coiled)

16 NUCOR STEEL **CRSI**
(Utah Mill in Plymouth, UT)

S

Bars #19 through #25 only

16 NUCOR STEEL **CRSI**
(Utah Mill in Plymouth, UT)

S

Bars #19 through #36 only

16 NUCOR STEEL **CRSI**
(Utah Mill in Plymouth, UT)

S

Bars #43 and #57 only

16 NUCOR STEEL **CRSI**
(Utah Mill in Plymouth, UT)

W

Bars #10 through #25 only (#13 and #16 coiled)

16 NUCOR STEEL **CRSI**
(Utah Mill in Plymouth, UT)

W

Bars #19 through #36 only

17 RIVERVIEW STEEL CORP.
(Glassport, PA)

S

Bars #10 through #19 only

18 ROCKY MOUNTAIN STEEL MILLS
(Pueblo, CO)

S

Coiled bars #3 through #5 only

18 ROCKY MOUNTAIN STEEL MILLS
(Pueblo, CO)

S

Coiled bars #3 through #7 only

Note: CRSI mill members are in boldface with the CRSI logo at the top right corner.

APPENDIX A

U.S. MANUFACTURERS OF GRADE 60 [420] CONCRETE REINFORCING BARS

ASTM and AASHTO Specifications require that all reinforcing bars be identified by permanent, mill imprinted markings.

18 ROCKY MOUNTAIN STEEL MILLS
(Pueblo, CO)

W

Coiled bars #3 through #5 only

18 ROCKY MOUNTAIN STEEL MILLS
(Pueblo, CO)

W

Coiled bars #3 through #7 only

19 SHEFFIELD STEEL CRSI
(Sand Springs, OK)

S

Bars #13 through #43 only

20 SILVER, INC., W.
(El Paso, TX)

I

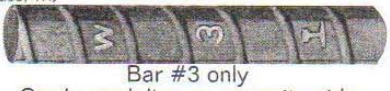
Bar #3 only

20 SILVER, INC., W.
(El Paso, TX)

I

Bar #3 only

20 SILVER, INC., W.
(El Paso, TX)

I 

Bar #3 only
Grade mark line on opposite side

21 STRUCTURAL METALS, INC. CRSI
(Arkansas Mill in Magnolia, AR)

S

Bars #10 through #19 only

21 STRUCTURAL METALS, INC. CRSI
(Arkansas Mill in Magnolia, AR)

I

Bars #10 through #19 only

21 STRUCTURAL METALS, INC. CRSI
(South Carolina Mill in Cayce, SC)

S

Bars #10 through #36 only

21 STRUCTURAL METALS, INC. CRSI
(South Carolina Mill in Cayce, SC)

S

Bars #43 and #57 only

21 STRUCTURAL METALS, INC. CRSI
(South Carolina Mill in Cayce, SC)

W

Bars #10 through #36 only

21 STRUCTURAL METALS, INC. CRSI
(South Carolina Mill in Cayce, SC)

W

Bars #43 and #57 only

21 STRUCTURAL METALS, INC. CRSI
(Texas Mill in Seguin, TX)

S

Bars #10 through #36 only

21 STRUCTURAL METALS, INC. CRSI
(Texas Mill in Seguin, TX)

S

Bars #43 and #57 only

21 STRUCTURAL METALS, INC. CRSI
(Texas Mill in Seguin, TX)

W

Bars #10 through #36 only

21 STRUCTURAL METALS, INC. CRSI
(Texas Mill in Seguin, TX)

W

Bars #43 and #57 only

22 TAMCO CRSI
(Rancho Cucamonga, CA)

S

Bars #13 through #36 only

22 TAMCO CRSI
(Rancho Cucamonga, CA)

S

Bars #43 and #57 only

Note: CRSI mill members are in boldface with the CRSI logo at the top right corner.

U.S. MANUFACTURERS OF GRADE 60 [420] CONCRETE REINFORCING BARS

ASTM and AASHTO Specifications require that all reinforcing bars be identified by permanent, mill imprinted markings.

22 TAMCO CRSI
(Rancho Cucamonga, CA)

W

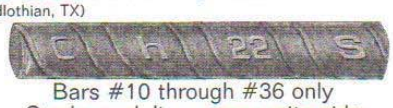
Bars #13 through #36 only

22 TAMCO CRSI
(Rancho Cucamonga, CA)

W

Bars #43 and #57 only

23 TXI CHAPARRAL STEEL
(Midlothian, TX)

S 

Bars #10 through #36 only
Grade mark line on opposite side

23 TXI CHAPARRAL STEEL
(Midlothian, TX)

W

Bars #10 through #36 only
Grade mark line on opposite side

Note: CRSI mill members are in boldface with the CRSI logo at the top right corner.

U.S. MANUFACTURERS OF GRADE 60 [420] CONCRETE REINFORCING BARS

CONTACT INFORMATION FOR CRSI MILL MEMBERS

2. **AMERISTEEL**
   Charlotte Steel Mill Division
   6601 Lakeview Rd
   Charlotte, NC 28213
   Tel: (704) 596-0361
   Fax: (704) 597-5031
   Web: www.ameristeel.com

   AMERISTEEL
   Jacksonville Steel Mill Division
   Hwy 217 & Yellow Water Rd
   Baldwin, FL 32234
   Tel: (904) 266-4261
   Fax: (904) 266-4244
   Web: www.ameristeel.com

   AMERISTEEL
   Knoxville Steel Mill Division
   1919 Tennessee Ave
   Knoxville, TN 37950
   Tel: (865) 546-5472
   Fax: (865) 637-8293
   Web: www.ameristeel.com

   AMERISTEEL
   West Tennessee Steel Mill Division
   801 AmeriSteel Rd
   Jackson, TN 38305
   Tel: (901) 424-5600
   Fax: (901) 422-4247
   Web: www.ameristeel.com

3. **AUBURN STEEL COMPANY, INC.**
   Auburn Division
   25 Quarry Rd
   Auburn, NY 13021
   Tel: (315) 253-4561
   Fax: (315) 253-5377
   Web: www.austeel.com

   AUBURN STEEL COMPANY, INC.
   Lemont Division
   New Ave at Ceco Rd
   Lemont, IL 60439
   Tel: (630) 243-0012
   Fax: (630) 243-0031
   Web: www.austeel.com

5. **BIRMINGHAM STEEL CORP.**
   Alabama Steel Division
   2301 Shuttlesworth Dr
   Birmingham, AL 35234
   Tel: (205) 252-8777
   Fax: (205) 250-7465
   Web: www.birminghamsteel.com

   BIRMINGHAM STEEL CORP.
   Illinois Steel Division, Joliet Rolling Mill
   927 Collins
   Joliet, IL 60432
   Tel: (815) 774-6145
   Fax: (815) 774-6105
   Web: www.birminghamsteel.com

   BIRMINGHAM STEEL CORP.
   Illinois Steel Division, Kankakee Plant
   972 East 4500 North Rd
   Bourbonnais, IL 69014
   Tel: (815) 937-3131
   Fax: (815) 939-5599
   Web: www.birminghamsteel.com

   BIRMINGHAM STEEL CORP.
   Mississippi Steel Division
   3630 Fourth St
   Jackson, MS 39208
   Tel: (601) 939-1623
   Fax: (601) 936-6200
   Web: www.birminghamsteel.com

   BIRMINGHAM STEEL CORP.
   Seattle Washington Steel Division
   2424 SW Andover
   Seattle, WA 98106
   Tel: (206) 933-2222
   Fax: (206) 933-2207
   Web: www.birminghamsteel.com

6. **BORDER STEEL, INC.**
   P.O. Box 12843
   El Paso, TX 79913
   Tel: (915) 886-2000
   Fax: (915) 886-2218
   Web: www.bordersteel.com

7. **CASCADE STEEL ROLLING MILLS, INC.**
   3200 NorthEast Highway 99W
   McMinnville, OR 97128
   Tel: (503) 472-4181
   Fax: (503) 434-5739
   Web: www.schn.com

U.S. MANUFACTURERS OF GRADE 60 [420] CONCRETE REINFORCING BARS

CONTACT INFORMATION FOR CRSI MILL MEMBERS

13. MARION STEEL COMPANY
912 Cheney Ave
Marion, OH 43302
Tel: (740) 383-4011
Fax: (740) 383-6429
Web: www.marionsteel.com

14. NORTH STAR STEEL COMPANY
Beaumont Mill
PO Box 2390
Beaumont, TX 77704
Tel: (409) 768-1211
Fax: (409) 769-1978
Web: www.cargillsteel.com/carnss

NORTH STAR STEEL COMPANY
Kingman Mill
3000 Hwy 66 South
Kingman, AZ 86413
Tel: (520) 718-0119
Fax: (520) 718-7093
Web: www.cargillsteel.com/carnss

NORTH STAR STEEL COMPANY
Monroe Mill
3000 East Front St
Monroe, MI 48161
Tel: (734) 243-2446
Fax: (734) 243-2751
Web: www.cargillsteel.com/carnss

NORTH STAR STEEL COMPANY
St. Paul Mill
1678 Red Rock Rd
St. Paul, MN 55119
Tel: (651) 731-5600
Fax: (651) 731-5699
Web: www.cargillsteel.com/carnss

NORTH STAR STEEL COMPANY
Wilton Mill
Highway 38 Greens Rd
Wilton, IA 52778
Tel: (319) 732-3231
Fax: (319) 732-4575
Web: www.cargillsteel.com/carnss

16. NUCOR STEEL
South Carolina Mill
P.O. Box 523
Darlington, SC 29540
Tel: (843) 393-5841
Fax: (843) 395-8701
Web: www.nucor.com

NUCOR STEEL
Texas Mill
P.O. Box 126
Jewett, TX 75846
Tel: (903) 626-4461
Fax: (903) 626-6262
Web: www.nucor.com

NUCOR STEEL
Utah Mill
P.O. Box 100
Plymouth, UT 84330
Tel: (435) 458-2300
Fax: (435) 458-2309
Web: www.nucor.com

19. SHEFFIELD STEEL CORP.
2300 South Hwy 97
Sand Springs, OK 74063
Tel: (918) 245-1335
Fax: (918) 245-9343
Web: www.sheffieldsteel.com

21. STRUCTURAL METALS, INC.
Arkansas Mill
PO Box 1147
Magnolia, AR 71753
Tel: (870) 234-8703
Fax: (870) 234-8706
Web: www.steelnet.org/cmc

STRUCTURAL METALS, INC.
South Carolina Mill
310 New State Rd
Cayce, SC 29033
Tel: (803) 936-3700
Fax: (803) 936-3711
Web: www.steelnet.org/cmc

STRUCTURAL METALS, INC.
Texas Mill
PO Box 911
Seguin, TX 78156
Tel: (830) 372-8200
Fax: (830) 379-9873
Web: www.steelnet.org/cmc

22. TAMCO
12459 Arrow Hwy
Rancho Cucamonga, CA 91739
Tel: (909) 899-0660
Fax: (909) 899-1910

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Reinforcing Steel Bar Chart

The attached chart contains bar sizes that reflects both English and Metric (SI) units of the ASTM reinforcing bar standards.

Please note, A615 and A706 specifications are currently in use; the A305 specification existed from 1947 to 1968. For older structures, it is prudent to anticipate varieties of reinforcing bars such as square bars.

REINFORCING STEEL

ASTM STANDARD REINFORCING BARS

BAR SIZE		NOMINAL DIMENSIONS - ROUND SECTIONS				APPROX DIAMETER OUTSIDE DEFORMATIONS (inches)
ENGLISH	METRIC	CROSS SECTIONAL AREA (in^2)	WEIGHT (lbs/ft)	NOMINAL DIAMETER (inches)	PERIMETER (inches)	
# 3	[# 10]	0.11	0.376	0.375	1.178	0.4375
# 4	[# 13]	0.20	0.668	0.500	1.571	0.5625
# 5	[# 16]	0.31	1.043	0.625	1.963	0.6875
# 6	[# 19]	0.44	1.502	0.750	2.356	0.875
# 7	[# 22]	0.60	2.044	0.875	2.749	1
# 8	[# 25]	0.79	2.670	1.000	3.142	1.125
# 9	[# 29]	1.00	3.400	1.128	3.544	1.25
# 10	[# 32]	1.27	4.303	1.270	3.99	1.4375
# 11	[# 36]	1.56	5.313	1.410	4.43	1.625
# 14	[# 43]	2.25	7.650	1.693	5.306	1.875
# 18	[# 57]	4.00	13.600	2.257	7.088	2.5

Reference: Concrete Reinforcing Steel Institute (CRSI) Manual of Standard Practice dated May 2003

Welded Wire Fabric

Shotcrete applications, slope paving, and other structure related items may require the use of welded wire fabric.

Chapter 2, "Welded Wire Fabric (WWF)", from the Concrete Reinforcing Institute, Manual of Standard Practices, 2009, provides information for specifying and estimating welded wire fabric and is included in Attachment No. 1 for Structure Construction personnel.

CHAPTER 2

WELDED WIRE FABRIC (WWF)

2.1. Introduction

This chapter presents information for specifying and estimating welded wire fabric (WWF) used in building construction. Discussion of epoxy-coated WWF, and handling, shipping, unloading and placing of WWF are also included.

2.2. ASTM Specifications

Welded wire fabric consists of wires arranged in a square or rectangular configuration. The wires are welded at their intersections. WWF must conform to ASTM A185 if made of plain wire or to ASTM A497 if made of deformed wire or a combination of deformed and plain wire. These specifications require tensile, reduction of area and bend tests on the fabric, and shear tests on the welded intersections. A minimum yield strength of 65,000 psi [450 MPa] is required for plain WWF (A185) and a minimum of 70,000 psi [485 MPa] for deformed WWF (A497). ASTM A82 (plain wire) and A496 (deformed wire) are companion specifications that prescribe the requirements for the wire used for manufacturing welded wire fabric.

Unless otherwise specified by the Architect/Engineer, welded wire fabric conforming to ASTM A185 will be furnished.

Welded wire fabric can be produced with high-strength wires of minimum yield strengths to 80,000 psi [550 MPa]. Higher minimum yield strengths allow the use of less material in certain applications.

Welded wire fabric can be fabricated to make beam stirrups and column ties.

2.3. Style Identification

Plain wire is denoted by the letter "W" ["MW"] and deformed wire by the letter "D" ["MD"]. The letter is followed by a number indicating cross-sectional area in hundredths of a square inch [square millimeters].

Welded wire fabric is usually shown on project drawings with the abbreviation WWF followed by spacings of longitudinal wires and then transverse wires and last by the sizes of longitudinal and transverse wires.

An example style designation (see Figure 2-1) is: WWF 6 × 12—W16 × W8 [152 × 305 – MW103 × MW52]. This designation identifies a style of plain welded wire fabric in which:

Spacing of longitudinal wires	= 6 in.	[152 mm]
Spacing of transverse wires	= 12 in.	[305 mm]
Longitudinal wire size	= W16	[MW103]
Transverse wire size	= W8	[MW52]

A deformed WWF style would be designated in the same manner with the appropriate D [MD] number wire spacings and sizes.

It is important to note that the terms "longitudinal" and "transverse" are related to the method of WWF manufacture and have no reference to the orientation of the wires with respect to the orientation of the reinforced concrete structure.

2.4. Specifying Welded Wire Fabric

The Architect/Engineer's selection of welded wire fabric styles should include production considerations as well as steel area requirements. Maximum economies in production and handling can be achieved by utilizing repetition of styles and duplication of sheet and/or roll dimensions to the fullest extent possible.

Welded wire fabric is manufactured in the form of sheets and rolls. Rolls are generally stocked in W1.4 to W4 [MW9 to MW26] wire sizes only. Roll widths vary from 5 to 8 feet [1.5 to 2.4 m]. Lengths vary with application and convenience of handling and shipping. Rolls should be straightened. Standard widths of sheets vary between 7 to 10 feet [2.1 to 3.1 m] for building construction and up to 13 feet [4 m] for pavement.

The maximum sheet size (width and/or length) may be limited by shipping restrictions as well as manufacturing limitations.

Development lengths and lap splice lengths for welded wire fabric must be specified by the Architect /Engineer in accordance with the ACI 318 Building Code. Lap splice lengths are usually a minimum of one wire space plus 2 in. [50 mm] for plain wire and 8 in. [200 mm] for deformed wire.

Certain styles of welded wire fabric as shown in Table 2-1 have been recommended by the Wire Reinforcement Institute as common styles. Manufacturers of WWF can meet specific steel area requirements when ordered for designated projects, or in some localities, may be available from inventory.

2.5. Detailing Welded Wire Fabric

The quantity of welded wire fabric detailed and supplied should include the net area shown on the project drawings or required in the project specifications plus sufficient material to include lap splices.

2.5.1 Width

Width is defined as the center-to-center distance between the outside longitudinal wires. Overall width is defined as the width plus side overhangs.

WELDED WIRE FABRIC (WWF)

The side overhangs of transverse wires should be no greater than one inch [25 mm] unless otherwise specified by the Architect/Engineer. Transverse wires may be specified to have a specific overhang or no overhang (flush sides).

2.5.2 Length

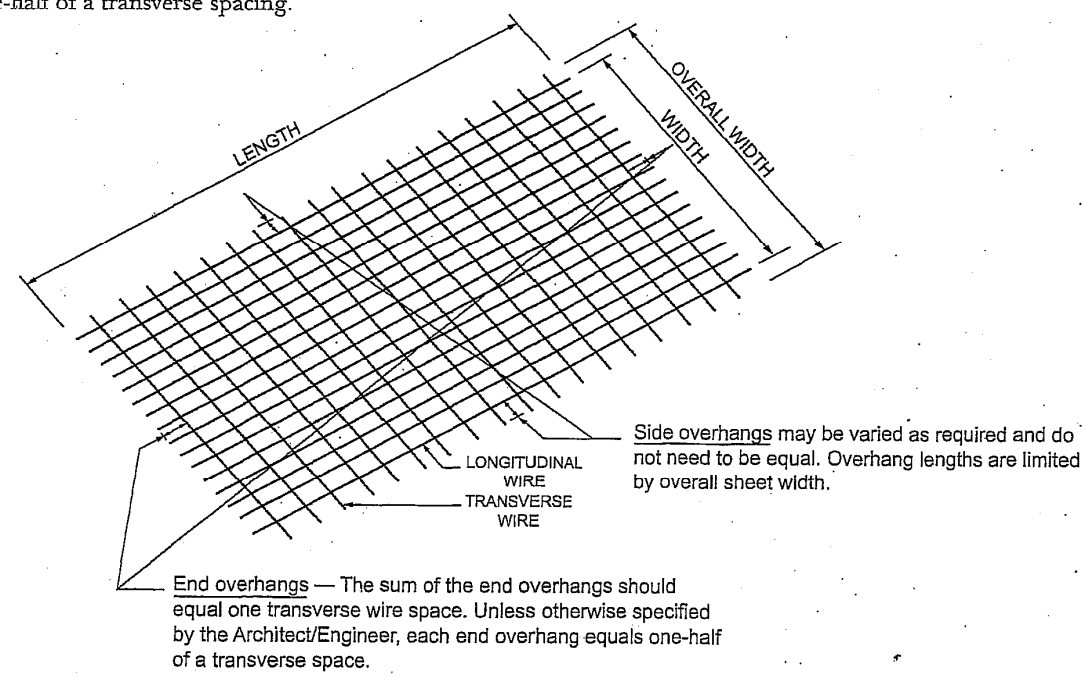
Welded wire fabric in roll form can be manufactured in various lengths, up to the maximum weight per roll convenient for handling. The lengths of rolls vary with the individual manufacturing practices of producers. Typical lengths are 100, 150 and 200 feet [31, 46 and 61 m]. Sheet or roll length is defined as the length, tip to tip, of longitudinal wires. This length should be a whole multiple of the transverse wire spacing.

The sum of the two end overhangs on either sheets or rolls should be equal to one transverse wire spacing. Unless otherwise specified, each end overhang equals one-half of a transverse spacing.

2.6. ASTM Specification for Epoxy-Coated Wire and Welded Wire Fabric

Epoxy-coated wire and welded wire fabric are used in reinforced concrete construction as a corrosion-protection system.

The ASTM specification A884/A884M covers the epoxy coating of plain and deformed steel wire, and plain and deformed steel welded wire fabric. The specification includes requirements for the epoxy-coating material; surface preparation of the steel prior to application of the coating; the method of application of the coating; limits on coating thickness; and acceptance tests to ensure that the coating was properly applied. All damaged areas of coating on the wires, which occur during manufacture and handling to the point of shipment to the job-site, have to be repaired (touched-up) with patching material.



Industry Method of Designating Style:
Example: WWF 6 x 12 — W16 x W8 [152 x 305 — MW103 x MW52]

Longitudinal wire spacing . . . . . 6 in. [152 mm]	Longitudinal wire size . . . . W16 [MW103]
Transverse wire spacing . . . . 12 in. [305 mm]	Transverse wire size . . . . . W8 [MW52]

WELDED WIRE FABRIC (WWF)

TABLE 2-1 COMMON STYLES OF WELDED WIRE FABRIC SHEETS

Inch-Pound Units			Metric Units		
Style	Area (in.²/ft)	Weight* (lb/100 ft²)	Style	Area (mm²/m)	Mass* (kg/m²)
4 x 4 – W1.4 x W1.4**	0.042	31	102 x 102 – MW9 x MW9**	88.9	1.51
4 x 4 – W2.0 x W2.0**	0.060	44	102 x 102 – MW13 x MW13**	127.0	2.15
4 x 4 – W2.9 x W2.9**	0.087	62	102 x 102 – MW19 x MW19**	184.2	3.03
4 x 4 – W3.1 x W3.1	0.093	65	102 x 102 – MW20 x MW20	196.9	3.17
4 x 4 – W4.0 x W4.0**	0.120	88	102 x 102 – MW26 x MW26**	254.0	4.30
6 x 6 – W1.4 x W1.4**	0.028	21	152 x 152 – MW9 x MW9 **	59.3	1.03
6 x 6 – W2.0 x W2.0**	0.040	30	152 x 152 – MW13 x MW13**	84.7	1.46
6 x 6 – W2.9 x W2.9**	0.058	42	152 x 152 – MW19 x MW19**	122.8	2.05
6 x 6 – W4.0 x W4.0**	0.080	58	152 x 152 – MW26 x MW26 **	169.4	2.83
6 x 6 – W4.2 x W4.2	0.084	60	152 x 152 – MW27 x MW27	177.8	3.08
6 x 6 – W4.4 x W4.4	0.088	63	152 x 152 – MW28 x MW28	186.3	3.22
6 x 6 – W4.7 x W4.7	0.094	68	152 x 152 – MW30 x MW30	199.0	3.32
6 x 6 – W7.5 x W7.5	0.150	108	152 x 152 – MW48 x MW48	317.5	5.52
6 x 6 – W8.1 x W8.1	0.162	116	152 x 152 – MW52 x MW52	342.9	5.66
6 x 6 – W8.3 x W8.3	0.166	119	152 x 152 – MW54 x MW54	351.4	5.81
12 x 12 – W8.3 x W8.3	0.083	63	305 x 305 – MW54 x MW54	175.7	3.08
12 x 12 – W8.8 x W8.8	0.088	66	305 x 305 – MW57 x MW57	186.3	3.22
12 x 12 – W9.1 x W9.1	0.091	69	305 x 305 – MW59 x MW59	192.6	8.25
12 x 12 – W9.4 x W9.4	0.094	71	305 x 305 – MW61 x MW61	199.0	3.47
12 x 12 – W15 x W15	0.150	113	305 x 305 – MW97 x MW97	317.5	5.52
12 x 12 – W16 x W16	0.160	120	305 x 305 – MW103 x MW103	338.7	5.61
12 x 12 – W16.6 x W16.6	0.166	125	305 x 305 – MW107 x MW107	351.4	9.72
12 x 12 – W17.1 x W17.1	0.171	128	305 x 305 – MW110 x MW110	362.0	6.25

* Weight [mass] based on 60-in. [1524-mm] wide sheets (c.-c.) with 1-in. [25-mm] side overhang and standard end overhang.

** These styles may be obtained in roll form. It is recommended that rolls be straightened and cut to size before placement.

Example Calculations:

6 x 6 – W4.0 x W4.0
Long. Wires (Table 2-2(a)) = 29.92
Tran. Wires (Table 2-2(c)) = 28.11
Total = 58.03 = 58 lb/100 ft²

152 x 152 – MW52 x MW52
Long. Wires (Table 2-2(b)) = 2.92
Tran. Wires (Table 2-2(d)) = 2.74
Total = 5.66 kg/m²

WELDED WIRE FABRIC (WWF)

TABLE 2-2(a) UNIT WEIGHT OF LONGITUDINAL WIRES FOR WELDED WIRE FABRIC (INCH-POUND)

Wire Size, W or D	Nom. Diam. (in.)	Weight (lb/100 ft²)* of Longitudinal Wires Per Spacing (in.)								
		2	3	4	5	6	8	9	10	12
45	0.757	948.60	642.60	489.60	397.80	336.60	260.10	234.40	214.20	183.60
31	0.628	653.40	442.68	337.28	274.04	231.88	179.18	161.68	147.56	126.48
30	0.618	632.40	428.40	326.40	265.20	224.40	173.40	156.46	142.80	122.40
28	0.597	590.24	399.84	304.64	247.52	209.44	161.84	146.03	133.28	114.24
26	0.575	548.08	371.28	282.88	229.84	194.48	150.28	135.60	123.76	106.08
24	0.553	505.92	342.72	261.12	212.16	179.52	138.72	125.17	114.24	97.92
22	0.529	463.76	314.16	239.36	194.48	164.56	127.16	114.74	104.72	89.76
20	0.505	421.60	285.60	217.60	176.80	149.60	115.60	104.31	95.20	81.60
18	0.479	379.44	257.04	195.84	159.12	134.64	104.04	93.88	85.68	73.44
16	0.451	337.28	228.48	174.48	141.44	119.68	92.48	83.45	76.16	65.28
14	0.422	295.12	199.92	152.32	123.76	104.72	80.92	73.01	66.64	57.12
12	0.391	252.96	171.36	130.56	106.08	89.76	69.36	62.58	57.12	48.96
11	0.374	231.88	157.08	119.68	97.24	82.28	63.58	57.37	52.36	44.88
10.5	0.366	221.34	149.94	114.24	92.82	78.54	60.69	54.76	49.98	42.84
10	0.357	210.80	142.80	108.80	88.40	74.80	57.80	52.15	47.60	40.80
9.5	0.348	200.26	135.66	103.36	83.98	71.06	54.91	49.55	45.22	38.76
9	0.339	189.72	128.52	97.92	79.56	67.32	52.02	46.94	42.84	36.72
8.5	0.329	179.18	121.38	92.48	75.14	63.58	49.13	44.33	40.46	34.68
8	0.319	168.64	114.24	87.04	70.72	59.84	46.24	41.73	38.08	32.64
7.5	0.309	158.10	107.10	81.60	66.30	56.10	43.35	39.11	35.70	30.60
7	0.299	147.56	99.96	76.16	61.88	52.36	40.46	36.51	33.32	28.56
6.5	0.288	137.02	92.82	70.72	57.46	48.62	37.57	33.90	30.94	26.52
6	0.276	126.48	85.68	65.28	53.04	44.88	34.68	31.29	28.56	24.48
5.5	0.265	115.94	78.54	69.84	48.62	41.14	31.79	28.69	26.18	22.44
5	0.252	105.40	71.40	54.40	44.20	37.40	28.90	36.08	23.80	20.40
4.5	0.239	94.86	64.26	48.96	39.78	33.66	26.01	23.47	21.42	18.36
4	0.226	84.32	57.12	43.52	35.36	29.92	23.12	20.87	19.04	16.32
3.5	0.211	73.78	49.98	38.08	30.94	26.18	20.23	18.26	16.66	14.28
3	0.195	63.24	42.84	32.64	26.52	22.44	17.34	15.65	14.28	12.24
2.9	0.192	61.13	41.41	31.55	25.64	21.69	16.76	15.11	13.80	11.83
2.5	0.178	52.70	35.70	27.20	22.10	18.70	14.45	13.04	11.90	10.20
2	0.160	42.16	28.56	21.76	17.68	14.96	11.56	10.44	9.52	8.16
1.4	0.134	29.51	19.99	15.23	12.38	10.47	8.09	7.29	6.66	5.71

*Weight based on standard end overhang.

Note: This table should be used for estimating purposes only. Actual weights of welded wire fabric will vary from those shown above, depending upon the width of rolls or sheets and lengths of overhangs. No allowance is made in this table for the extra weight of fabric required for lap splices.

WELDED WIRE FABRIC (WWF)

TABLE 2-2(b) UNIT MASS OF LONGITUDINAL WIRES FOR WELDED WIRE FABRIC (METRIC)

Wire Size, MW or MD	Nom. Diam. (mm)	Mass (kg/m²)* of Longitudinal Wires Per Spacing (mm)								
		51	76	102	127	152	203	229	254	305
290	19.23	46.31	31.37	23.90	19.42	16.43	12.71	11.44	10.46	8.96
200	15.96	31.94	21.59	16.45	13.37	11.31	8.74	7.89	7.20	6.17
194	15.70	30.84	20.89	15.92	12.93	10.94	8.36	7.63	6.96	5.97
181	15.17	28.79	19.50	14.86	12.07	10.22	7.89	7.12	6.50	5.57
168	14.61	26.73	18.11	13.80	11.21	9.49	7.33	6.61	6.04	5.17
155	14.04	24.68	16.72	12.74	10.35	8.76	6.77	6.15	5.57	4.78
142	13.44	23.60	15.32	11.67	9.49	8.03	6.20	5.60	5.11	4.38
129	12.82	20.56	13.94	10.61	8.62	7.30	5.84	5.09	4.64	3.98
116	12.16	18.51	12.54	9.55	7.76	6.57	5.07	4.58	4.23	3.58
103	11.46	16.45	11.14	8.49	6.90	5.84	4.51	4.07	3.71	3.18
90	10.72	14.39	9.75	7.43	6.04	5.11	3.95	3.55	3.25	2.79
77	9.93	12.34	8.38	6.37	6.17	4.38	3.38	3.05	2.79	2.39
71	9.51	11.31	7.66	5.81	4.74	4.01	3.10	2.80	2.55	2.19
68	9.29	10.80	7.31	5.57	4.53	3.83	2.96	2.67	2.44	2.09
65	9.06	10.27	6.96	5.31	4.31	3.65	2.82	2.54	2.32	1.99
61	8.83	9.77	6.62	5.04	4.10	3.47	2.68	2.42	2.21	1.89
58	8.60	9.25	6.27	4.78	3.88	3.28	2.54	2.29	2.09	1.79
55	8.36	8.74	5.82	4.56	3.66	3.10	2.40	2.16	1.97	1.69
52	8.11	8.23	5.57	4.25	3.45	2.92	2.26	2.04	1.86	1.59
48	7.85	7.71	5.22	3.98	3.23	2.74	2.11	1.91	1.74	1.49
45	7.58	7.20	4.88	3.71	3.02	2.55	1.97	1.78	1.63	1.39
42	7.31	6.68	4.53	3.45	2.80	2.37	1.83	1.65	1.51	1.29
39	7.02	6.17	4.18	3.18	2.59	2.19	1.69	1.53	1.39	1.19
35	6.72	5.65	3.83	2.92	2.37	2.01	1.55	1.40	1.28	1.09
32	6.41	5.14	3.48	2.65	2.18	1.82	1.41	1.27	1.18	0.99
29	6.08	4.63	3.13	2.39	1.94	1.64	1.27	1.14	1.04	0.90
26	5.73	4.11	2.79	2.12	1.72	1.46	1.13	1.02	0.93	0.80
23	5.36	3.60	2.44	1.86	1.51	1.28	0.99	0.89	0.81	0.70
19.4	4.96	4.06	2.09	1.59	1.29	1.09	0.85	0.76	0.70	0.60
18.7	4.88	2.98	2.02	1.54	1.25	1.06	0.82	0.74	0.67	0.58
16	4.53	2.57	1.74	1.33	1.08	0.91	0.70	0.64	0.58	0.50
13	4.05	2.06	1.39	1.03	0.87	0.73	0.56	0.51	0.46	0.40
9	3.39	1.44	0.98	0.74	0.60	0.51	0.40	0.36	0.33	0.28

*Mass based on standard end overhang.

Note: This table should be used for estimating purposes only. Actual mass of welded wire fabric will vary from those shown above, depending upon the width of rolls or sheets and lengths of overhangs. No allowance is made in this table for the extra mass of fabric required for lap splices.

WELDED WIRE FABRIC (WWF)

TABLE 2-2(c) UNIT WEIGHT OF TRANSVERSE WIRES FOR WELDED WIRE FABRIC (INCH-POUND)

Wire Size, W or D	Nom. Diam. (in.)	Weight (lb/100 ft²)* of Transverse Wires Per Spacing (in.)								
		2	3	4	5	6	8	9	10	12
45	0.757	948.57	632.38	474.29	379.43	316.19	237.14	210.79	189.72	158.10
31	0.628	653.48	435.65	326.74	261.39	217.83	163.37	145.22	130.70	108.91
30	0.618	632.40	421.40	316.20	252.96	210.80	158.10	140.53	126.48	105.40
28	0.597	590.24	393.49	295.12	236.10	196.75	147.56	131.17	118.05	98.37
26	0.575	548.08	365.38	274.04	219.23	182.70	137.02	121.80	109.62	91.34
24	0.553	505.92	337.28	252.96	202.37	168.64	126.48	112.43	101.18	84.32
22	0.529	463.76	309.17	231.88	185.50	154.59	115.94	103.06	92.75	77.29
20	0.505	421.60	281.06	210.80	168.64	140.53	105.40	93.69	84.32	70.26
18	0.479	379.44	252.96	189.72	151.78	126.48	94.86	84.32	75.89	63.24
16	0.451	337.28	224.85	168.64	134.91	112.43	84.32	74.95	67.46	56.21
14	0.422	295.12	196.76	147.56	118.05	98.37	73.78	65.58	59.02	49.19
12	0.391	252.96	168.64	126.48	101.18	84.32	63.24	56.21	50.59	42.16
11	0.374	231.88	154.59	115.94	92.75	77.29	57.97	51.53	46.38	38.65
10.5	0.366	221.34	147.56	110.67	88.54	73.78	55.34	49.19	44.27	36.89
10	0.357	210.80	140.53	105.40	84.32	70.27	52.70	46.84	42.16	35.13
9.5	0.348	200.28	133.51	100.13	80.11	66.76	50.07	44.50	40.05	33.38
9	0.339	189.72	126.48	94.86	75.89	63.24	47.43	42.16	37.94	31.62
8.5	0.329	179.18	119.45	89.59	71.67	59.73	44.80	39.82	35.84	29.86
8	0.319	168.64	112.43	84.32	67.46	56.21	42.16	37.48	33.73	28.11
7.5	0.309	158.10	105.40	79.05	63.24	52.70	39.53	35.14	31.62	26.35
7	0.299	147.56	98.37	73.78	59.02	49.19	36.89	32.79	29.51	24.59
6.5	0.288	137.02	91.35	68.51	54.81	45.68	34.26	30.45	27.41	22.84
6	0.276	126.48	84.32	63.24	50.59	42.16	31.62	28.11	25.30	21.08
5.5	0.265	115.94	77.30	57.97	46.38	38.65	28.99	25.77	23.19	19.33
5	0.252	105.40	70.27	52.70	42.16	35.13	26.35	23.42	21.08	17.57
4.5	0.239	94.86	63.24	47.43	37.95	31.62	23.72	21.08	18.97	15.81
4	0.226	84.32	56.21	42.16	33.73	28.11	21.08	18.74	16.86	14.05
3.5	0.211	73.78	49.19	36.89	29.51	24.60	18.45	16.40	14.76	12.30
3	0.195	63.24	42.16	31.62	25.30	21.08	15.81	14.05	12.65	10.54
2.9	0.192	61.13	40.75	30.56	24.45	20.38	15.28	13.58	12.23	10.19
2.5	0.178	52.70	35.13	26.35	21.08	17.57	13.18	11.71	10.54	8.78
2	0.160	42.16	28.11	21.08	16.86	14.05	10.54	9.37	8.43	7.03
1.4	0.134	29.51	19.67	14.76	11.80	9.84	7.38	6.56	5.90	4.92

* Weight based on 60-in. wide sheets (c.-c.) with 1-in. side overhang.

Note: This table should be used for estimating purposes only. Actual weights of welded wire fabric will vary from those shown above, depending upon the width of rolls or sheets and lengths of overhangs. No allowance is made in this table for the extra weight of fabric required for lap splices.

WELDED WIRE FABRIC (WWF)

TABLE 2-2(d) UNIT MASS OF TRANSVERSE WIRES FOR WELDED WIRE FABRIC (METRIC)

Wire Size, MW or MD	Nom. Diam. (mm)	Mass (kg/m²)* of Transverse Wires Per Spacing (mm)								
		51	76	102	127	152	203	229	254	305
290	19.23	45.72	30.48	22.86	17.91	15.24	11.43	9.91	9.14	7.62
200	15.96	32.36	21.25	15.94	12.75	10.62	7.97	7.08	6.37	5.31
194	15.70	30.84	20.56	15.52	12.34	10.28	7.71	6.85	6.17	5.14
181	15.17	28.79	19.19	14.44	11.52	9.60	7.20	6.40	5.76	4.80
168	14.61	26.73	17.82	13.37	10.69	8.91	6.68	5.94	5.35	4.46
155	14.04	24.88	16.45	12.34	9.87	8.23	6.17	5.48	4.93	4.11
142	13.44	22.62	15.08	11.31	9.06	7.54	5.65	5.03	4.52	3.77
129	12.82	20.66	13.71	10.28	8.23	6.85	5.14	4.57	4.11	3.43
116	12.16	18.51	12.34	9.25	7.40	6.17	4.63	4.11	3.70	3.08
103	11.46	16.45	10.97	8.23	6.58	5.48	4.11	3.66	3.29	2.74
90	10.72	14.39	9.60	7.20	5.76	4.80	3.60	3.20	2.88	2.40
77	9.93	12.34	8.23	6.17	4.93	4.11	3.08	2.74	2.47	2.08
71	9.51	11.31	7.54	5.65	4.52	3.77	2.83	2.52	2.26	1.89
68	9.29	10.80	7.20	5.40	4.32	3.60	2.70	2.40	2.16	1.80
65	9.06	10.28	6.85	5.14	4.11	3.43	2.57	2.28	2.06	1.71
61	8.83	9.77	6.51	4.88	3.91	3.26	2.44	2.17	1.95	1.63
58	8.60	9.25	6.17	4.63	3.70	3.08	2.31	2.06	1.85	1.54
55	8.36	8.74	5.83	4.37	3.50	2.91	2.19	1.94	1.75	1.46
52	8.11	8.24	5.48	4.11	3.29	2.74	2.06	1.83	1.65	1.37
48	7.85	7.71	5.14	3.86	3.08	2.57	1.93	1.71	1.55	1.29
45	7.58	7.20	4.80	3.60	2.88	2.40	1.80	1.60	1.44	1.20
42	7.31	6.68	4.45	3.34	2.67	2.23	1.67	1.49	1.34	1.11
39	7.02	6.17	4.11	3.08	2.47	2.00	1.54	1.37	1.23	1.03
35	6.72	5.65	3.77	2.83	2.26	1.89	1.41	1.26	1.13	0.94
32	6.41	5.14	3.43	2.57	2.06	1.71	1.29	1.14	1.03	0.86
29	6.08	4.63	3.08	2.31	1.85	1.54	1.16	1.03	0.93	0.77
26	5.73	4.11	2.74	2.06	1.65	1.27	1.03	0.91	0.82	0.69
23	5.36	3.60	2.40	1.80	1.44	1.20	0.90	0.80	0.72	0.60
19.4	4.96	3.08	2.08	1.54	1.23	1.03	0.77	0.69	0.62	0.51
18.7	4.88	2.95	1.96	1.47	1.15	0.98	0.74	0.64	0.59	0.49
16	4.53	2.57	1.76	1.29	1.03	0.86	0.64	0.57	0.51	0.43
13	4.05	2.06	1.37	1.03	0.81	0.69	0.51	0.46	0.41	0.34
9	3.39	1.42	0.95	0.71	0.56	0.47	0.36	0.31	0.28	0.24

* Mass based on 1524-mm wide sheets (c.-c.) with 25-mm side overhang.

Note: This table should be used for estimating purposes only. Actual mass of welded wire fabric will vary from those shown above, depending upon the width of rolls or sheets and lengths of overhangs. No allowance is made in this table for the extra mass of fabric required for lap splices.

WELDED WIRE FABRIC (WWF)

TABLE 2-3(a) CROSS-SECTIONAL AREA AND WEIGHT OF WELDED WIRE FABRIC (INCH-POUND)

Wire Size, W or D	Nom. Diam. (in.)	Nom. Weight (lb/ft)	Area of Steel (in.²/ft) Per Wire Spacing (in.)						
			2	3	4	6	8	10	12
45	0.757	1.530	2.700	1.800	1.350	0.900	0.675	0.540	0.450
31	0.628	1.054	1.860	1.240	0.930	0.620	0.465	0.372	0.310
30	0.618	1.020	1.800	1.200	0.900	0.600	0.450	0.360	0.300
28	0.597	0.952	1.680	1.120	0.840	0.560	0.420	0.336	0.280
26	0.575	0.884	1.560	1.040	0.780	0.520	0.390	0.312	0.260
24	0.553	0.816	1.440	0.960	0.720	0.480	0.360	0.288	0.240
22	0.529	0.748	1.320	0.880	0.660	0.440	0.330	0.264	0.220
20	0.505	0.680	1.200	0.800	0.600	0.400	0.300	0.240	0.200
18	0.479	0.612	1.080	0.720	0.540	0.360	0.270	0.216	0.180
16	0.451	0.544	0.960	0.640	0.480	0.320	0.240	0.192	0.160
14	0.422	0.476	0.840	0.560	0.420	0.280	0.210	0.168	0.140
12	0.391	0.408	0.720	0.480	0.360	0.240	0.180	0.144	0.120
11	0.374	0.374	0.660	0.440	0.330	0.220	0.165	0.132	0.110
10.5	0.366	0.357	0.630	0.420	0.315	0.210	0.158	0.126	0.105
10	0.357	0.340	0.600	0.400	0.300	0.200	0.150	0.120	0.100
9.5	0.348	0.323	0.570	0.380	0.285	0.190	0.143	0.114	0.095
9	0.339	0.306	0.540	0.360	0.270	0.180	0.135	0.108	0.090
8.5	0.329	0.289	0.510	0.340	0.255	0.170	0.128	0.102	0.085
8	0.319	0.272	0.480	0.320	0.240	0.160	0.120	0.096	0.080
7.5	0.309	0.255	0.450	0.300	0.225	0.150	0.113	0.090	0.075
7	0.299	0.238	0.420	0.280	0.210	0.140	0.105	0.084	0.070
6.5	0.288	0.221	0.390	0.260	0.195	0.130	0.098	0.078	0.065
6	0.276	0.204	0.360	0.240	0.180	0.120	0.090	0.072	0.060
5.5	0.265	0.187	0.330	0.220	0.165	0.110	0.083	0.066	0.055
5	0.252	0.170	0.300	0.200	0.150	0.100	0.075	0.060	0.050
4.5	0.239	0.153	0.270	0.180	0.135	0.090	0.068	0.054	0.045
4	0.226	0.136	0.240	0.160	0.120	0.080	0.060	0.048	0.040
3.5	0.211	0.119	0.210	0.140	0.105	0.070	0.053	0.042	0.035
3	0.195	0.102	0.180	0.120	0.090	0.060	0.045	0.036	0.030
2.9	0.192	0.099	0.174	0.116	0.087	0.058	0.044	0.035	0.029
2.5	0.178	0.085	0.150	0.100	0.075	0.050	0.038	0.030	0.025
2	0.160	0.068	0.120	0.080	0.060	0.040	0.030	0.024	0.020
1.4	0.134	0.048	0.084	0.056	0.042	0.028	0.021	0.017	0.014

Notes:

1. The above listing of plain and deformed wire sizes represents wires normally selected to manufacture welded wire fabric to specific areas of reinforcement. Wire sizes other than those listed above, including larger sizes, may be available if the quantity required is sufficient to justify manufacture.
2. The nominal diameter of a deformed wire is equivalent to the diameter of a plain wire having the same weight per foot as the deformed wire.
3. The ACI Building Code requirements for tension development lengths and tension lap splice lengths of welded wire fabric are not included in this chapter. These design requirements are covered in *Reinforcement Anchorages and Splices* available from CRSI. For additional information, see *Manual of Standard Practice—Structural Welded Wire Fabric* and *Structural Detailing Manual*, both published by the Wire Reinforcement Institute.

WELDED WIRE FABRIC (WWF)

TABLE 2-3(b) CROSS-SECTIONAL AREA AND MASS OF WELDED WIRE FABRIC (METRIC)

Wire Size, MW or MD	Nom. Diam. (mm)	Nom. Mass (kg/m)	Area of Steel (mm²/m) Per Wire Spacing (mm)						
			51	76	102	152	203	254	305
290	19.23	2.277	5715	3810	2858	1905	1429	1143	953
200	15.96	1.569	3937	2625	1969	1312	984	787	656
194	15.70	1.518	3810	2540	1905	1270	952	762	635
181	15.17	1.417	3556	2371	1778	1185	889	711	593
168	14.61	1.316	3302	2201	1651	1101	826	660	550
155	14.04	1.214	3048	2032	1524	1016	762	610	508
142	13.44	1.113	2794	1863	1397	931	698	559	466
129	12.82	1.012	2540	1693	1270	847	635	508	423
116	12.16	0.911	2286	1524	1143	762	571	457	381
103	11.46	0.810	2032	1355	1016	677	508	406	339
90	10.72	0.708	1778	1185	889	593	445	356	296
77	9.93	0.607	1524	1016	762	508	381	305	254
71	9.51	0.557	1397	931	698	466	349	279	233
68	9.29	0.531	1333	889	667	444	333	267	222
65	9.06	0.506	1270	847	635	423	317	254	212
61	8.83	0.481	1206	804	603	402	302	241	201
58	8.60	0.455	1143	762	571	381	286	229	190
55	8.36	0.430	1080	720	540	360	270	216	180
52	8.11	0.405	1016	677	508	339	254	203	169
48	7.85	0.379	952	635	476	317	238	190	159
45	7.58	0.354	889	593	445	296	222	178	148
42	7.31	0.329	826	550	413	275	206	165	138
39	7.02	0.304	762	508	381	254	190	152	127
35	6.72	0.278	698	466	349	233	175	140	116
32	6.41	0.253	635	423	317	212	159	127	106
29	6.08	0.228	571	381	286	190	143	114	95
26	5.73	0.202	508	339	254	169	127	102	85
23	5.36	0.177	445	296	222	148	111	89	74
19.4	4.96	0.152	381	254	190	127	95	76	63
18.7	4.88	0.147	368	246	184	123	92	74	61
16	4.53	0.126	317	212	159	106	79	63	53
13	4.05	0.101	254	169	127	85	63	51	42
9	3.39	0.071	178	119	89	59	44	36	30

Notes:

1. The above listing of plain and deformed wire sizes represents wires normally selected to manufacture welded wire fabric to specific areas of reinforcement. Wire sizes other than those listed above, including larger sizes, may be available if the quantity required is sufficient to justify manufacture.
2. The nominal diameter of a deformed wire is equivalent to the diameter of a plain wire having the same mass per foot as the deformed wire.
3. The ACI Building Code requirements for tension development lengths and tension lap splice lengths of welded wire fabric are not included in this chapter. These design requirements are covered in *Reinforcement Anchorages and Splices* available from CRSI. For additional information, see *Manual of Standard Practice—Structural Welded Wire Fabric* and *Structural Detailing Manual*, both published by the Wire Reinforcement Institute.

Reinforcement Splices

This memo provides general contract administration guidance for service splice and ultimate butt splice of reinforcement in accordance with the 2006 Standard Specifications (Amended Section 52) and the 2010 Standard Specifications. Sections included are:

- Glossary of Terms.
- What To Do Prior To the Start of Any Splice Work.
- What To Do During Splice Production Work.
- Splice Acceptance Requirements.
- Mechanical Splice Acceptance Procedure Flow Chart.
- Review Time.
- What to Do After Completion of Splice Production Work.
- Testing Requirement Clarifications for Welded Butt Splices.
- Items To Be Recorded In the Job Files.

Glossary of Terms

Affected Zone – Portion of the reinforcing bar where any properties of the bar, including the physical, metallurgical, or materials characteristics, have been changed by fabrication or installation of a splice. The weld and one (1) inch adjacent to the weld is part of the affected zone.

Authorized Laboratory – Independent testing laboratory not employed or compensated by any subcontractor or subcontractor's affiliate providing other services for the Contract and authorized by the Department to perform the required testing of the sample splices.

Authorized Material List - A list of products prequalified for use on California Department of Transportation projects. Mechanical couplers for both service and ultimate splice systems on this list are authorized for use.

Lot of Splices – One hundred and fifty (150) or fraction thereof, of the same coupler model for each bar size, deformation pattern and hoop diameter.

Operator and Procedure Prequalification – A requirement of the splice prequalification report. Splice operators and procedures must be certified. Splice test samples must be prepared and tested no more than two (2) years before the submittal.

Production Service Splice Test Samples – Four splices prepared in the same manner (i.e., equipment, procedures, position and operator) as the splices incorporated into the final work. Four samples are selected for every lot of splices and are tested by the authorized laboratory.

Production Ultimate Butt Splice Test Samples – Four splices removed from each lot of completed splices. After being notified, the Engineer randomly selects the four (4) splice test samples to be removed by the contractor from the completed lot and places tamper-proof markings or seals on them. Except for hoops, the Engineer selects splice test samples at the job site. Splice test samples must comply with California Test 670.

Quality Assurance (QA) Splice Samples – Four additional splice test samples prepared or removed for QA testing at the Caltrans Materials Engineering and Testing Services (METS) Laboratory. The samples are prepared or removed concurrently with the first production lot and at one other randomly selected for every five (5) additional production lots (or portion of) thereafter.

Splicing Quality Control Manager (QCM) – Contractor designated person who is responsible for both field and administrative work regarding the quality of all butt splices.

Resistance Butt Weld – A type of butt-welding commonly used to produce column hoop reinforcement. A machine holds both ends of the hoop together and passes a large electrical current through the bar which creates enough heat to fuse the two ends together completing the process.

This type of welding is not covered by the American Welding Society (AWS) code and therefore does not require any of the Non-Destructive Testing (NDT) or Certified Welding Inspector (CWI) requirements. The current specification requires that the fabricator must be on the Authorized Material List.

Service Splice – A mechanical or welded splice that meets the current requirements of the Standard Specifications (SS)[1] (i.e., tensile strength of 80 ksi and slip).

Splice Prequalification Report – A report that documents the Contractor's proposed splicing system.

Ultimate Butt Splice – A mechanical or welded butt splice that meets current requirements of SS[2] (i.e., slip test and rupture in the reinforcing bar outside of the affected zone and show visible necking as specified in California Test 670, Necking, Option I or Rupture anywhere and neck as specified in California Test 670, Necking, Option II).

[1] SS 2006, Sections 52-1.08B(1) and 52-1.08C(2), or SS 2010, Sections 52-6.02B and 52-6.02C.
[2] SS 2006, Sections 52-1.08B(1) and 52-1.08C(3), or SS 2010, Sections 52-6.02B and 52-6.02D.

What To Do Prior To the Start of Any Splice Work

Document Review:

The Structure Representative and/or his staff must conduct proper review of the following documents:

- Contract Special Provisions.
- Contract Plans.
- Standard Specifications, Section 52 (Reinforcement).
- Bridge Construction Records and Procedures Manual, Section 165
- Authorized Materials List for steel reinforcement splicing (found at http://www.dot.ca.gov/hq/esc/approved_products_list/).
- BCM 9-1.0 – Locating Reinforcing Steel Splices on As-Built Plans.
- California Test 670 – Method of Tests for Mechanical and Welded Reinforcing Steel Splices.

Timely Submittal of Required Documents

Ensure the Contractor provides timely submittals of the following documents as applicable:

- Certificate of Compliance.
- Welder and Welding Procedures Qualifications.
- Splice Prequalification Report.
- Weld Flash Removal Process (when welding is involved).

Notify METS

As soon as possible, inform your Structure Materials Representative (SMR) that your contract contains ultimate and or service reinforcing splices. Invite SMR to the pre-job meeting. The SMR contact list can be found at
http://www.dot.ca.gov/hq/esc/Translab/OSM/smdocuments/StructuralMaterialsRepresentatives.pdf

Splicing Quality Control Manager (QCM)

The contractor shall notify the Engineer of the name and contact information of the splicing QCM. Verify compliance of the assigned splicing QCM with the specification requirements and respond to the contractor accordingly.

Authorized Laboratory

Verify that the independent testing laboratory contracted by the Contractor to perform prequalification and acceptance testing is listed in the Authorized Laboratory List. The Authorized Laboratory List is available at
http://www.dot.ca.gov/hq/esc/Translab/authorized_laboratories_list/
If information is not available on the website, contact METS for assistance.

Pre-Job Meeting

If needed, a pre-job meeting with the contractor should be held to discuss the ultimate and service butt splice specification. The sampling and acceptance criteria are different for these types of splices. It is important that all parties involved understand the specification requirements.

At the meeting, the contractor should have present their splicing QCM, rebar subcontractor, and the representative for the Authorized Laboratory. If possible, a representative from METS should attend the pre-job meeting. A suggested partial list of items to discuss at the pre-job meeting is:

- Splicing QCM's responsibility to inspect the lots of splices for conformance with the specifications and manufacturer's recommendations prior to sampling.
- Splice Prequalification Reports, production and quality assurance (QA) sampling and testing requirements.
- How samples of ultimate splices will be selected from a completed lot of splices that have been assembled for the final time.
- Contractor's method of designating and making the lots available for sampling.
- Engineer's method of random sample selection.
- Labeling and shipping of the samples.
- Result reporting, time allowed, and Engineer approval.

Splice System Prequalification

The specifications require that both service and ultimate splice systems be prequalified, for every job, prior to use. The contractor must select a splice system from the Authorized Materials List. This list is posted at: http://www.dot.ca.gov/hq/esc/approved_products_list/ . If the proposed system is not on the prequalification list, contact METS at (916 227-7253) to verify the latest approved splice systems.

Splice Prequalification Report

For each splice type to be used in the work, the contractor must submit a Splice Prequalification Report for service splices and ultimate butt splices that includes:

- Copy of the coupler manufacturer's product literature giving complete data on the splice material and installation procedures.
- Names of the operators who will be performing the splicing.
- Descriptions of the positions, locations, equipment, and procedures that will be used in the work.
- Certifications from the fabricator for operator and procedure prequalification including the certified test results from the authorized laboratory for the prequalification splice test samples. For each bar size of each splice type to be used, each operator must prepare two (2) prequalification splice test samples and two (2) additional prequalification splice test samples if using splices dependent on bar deformations.
- Splice test samples must have been prepared and tested no more than two (2) years before the submittal of the splice prequalification report. Splice test samples and testing must comply with the production testing requirements.

What To Do During Splice Production Work

Inspection

During the production of both ultimate and service splices, some of the tasks that Caltrans/Structure Construction (SC) personnel should perform are:

- Verify the location of the splices. Note the splice locations on the As-Built plans.
- Check the material certification. Certificate of Compliance shall be provided for all material constituting the splice (i.e., reinforcing steel, coupler).
- Perform visual inspection of the production of ultimate or service splices.
- Randomly select production and quality assurance sample splices. This includes ensuring that the operators are prequalified and the equipment and procedures used conform to the manufacturer's recommendations.
- Confirm that all test reports meet the contract requirements.

Sampling of Production Splices
The sampling procedures and testing criteria are different for ultimate butt and service splices. Ultimate butt splices are far more critical to the structure's seismic performance. Hence, the sampling and testing requirements are more stringent compared to service splices.

Ultimate Splice Sampling Procedures
The contractor's splicing QCM will notify the Engineer when a designated lot of splices is complete and has been inspected. Four samples of production splices from each lot will be selected by the Engineer for testing.

Production sample splices are required to be randomly selected from a completed lot. Selecting from a completed lot means that samples will be removed after final splicing has been made. Splices that are unassembled for transportation or other reasons are not considered completed and would require resampling when assembled for the final time.

The intent of the ultimate butt splice specification is to sample splices as close as possible to the in place completed work, which may or may not entail removing splices from bars after they have been tied in their final location. For example, if the main longitudinal reinforcement of a column was spliced together and assembled on the ground prior to full height erection, the straight bar sample production splices could be selected prior to cage assembly. If splices are made vertically at the job site in or above their final positions for bar reinforcement of columns or CIP (cast-in-place) concrete piles, instead of removing the splice test samples from the completed lot, it is acceptable to prepare the samples as specified for service splice test samples provided testing as specified for ultimate butt splices is performed.

Similarly, in most cases, the selection of production samples for welded hoops can be done prior to cage assembly.

All production sample splices removed from the work must be repaired or replaced. The Department does not require ultimate butt splice testing on repaired splices from a lot unless an additional ultimate butt splice test is required on the same lot of splices. If this additional test is required, the Engineer may select any repaired splice for the additional test.

The sample length shall comply with California Test 670.

Service Splice Sampling Procedures
The specifications require the contractor to prepare four splice test samples from each lot of completed splices. The service splice samples must be prepared in the same conditions as the production service splices. The same operator, equipment, position, and procedures must be used when preparing service splice samples. The sample length shall comply with California Test 670.

Quality Assurance Testing
Quality assurance (QA) testing is a requirement of the ultimate butt and service splice specifications. Quality assurance tests are always performed concurrently with the first production test. Subsequent to the first QA test, at least one out of every five additional production tests (or portion of) thereafter will be accompanied by an additional QA test. A random selection method shall be used to designate both QA lots and QA sample splices. Below is a table that illustrates the number of QA tests required for a given amount of splice lots.

Number. of Lots	Number of Accumulative QA Tests Required
1	1
2-6	2
7-11	3
12-16	4
17-21	5
22-26	6

To obtain samples for the QA test, four (4) sample splices will be made concurrently with the production test samples. These sample splices do not have to be removed from a completed lot of splices.

The Contractor may encase splices in concrete before receiving notification of the QA test results from the Engineer. However, the Contractor will not be relieved of the responsibility for incorporating material into the work that complies with the Contract.

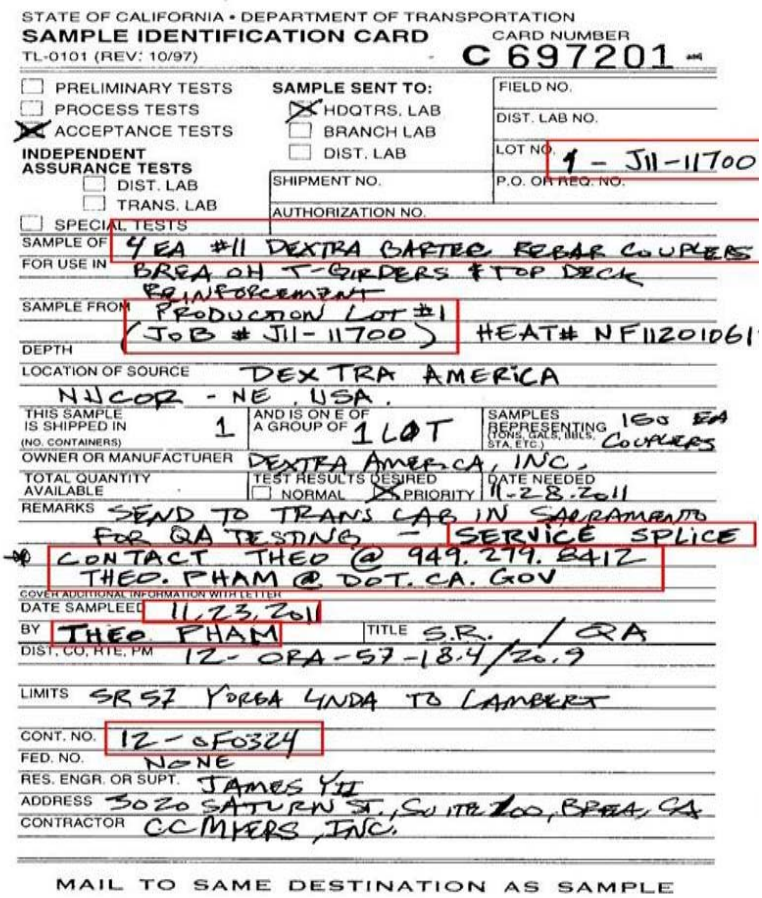
Tamper Proof Markings and Sample Shipping
To assure that the sample splices are not tampered with, all samples (i.e., pre-job, production, and quality assurance) shall have a tamper proof marking applied to them. Field personnel should apply the marking. Examples of tamper proof markings are:

- Rubberized paint. This will show any re-gripping or disassembly of the splices. See Figure No. 1.
- A digital photo of the splice sent to the lab for comparison.
- Alternative marking systems can be considered with METS concurrence.

All samples shall be indentified pre-job, production, or job QA and be accompanied with form TL-101. See Figure No. 2. Both pre-job and quality assurance ultimate butt splice test samples

need to be shipped to METS at 5900 Folsom Boulevard, Sacramento 95819, (916) 227-7251. The Structure Representative should discuss the method of shipment with METS.

Figure No. 1: Production Sample Splices with rubberized paint used as a tamper proof marking system. Associated control bars are no longer required by the specifications.



STATE OF CALIFORNIA • DEPARTMENT OF TRANSPORTATION
SAMPLE IDENTIFICATION CARD
TL-0101 (REV. 10/97)
CARD NUMBER C 697201

☐ PRELIMINARY TESTS
☐ PROCESS TESTS
☒ ACCEPTANCE TESTS
INDEPENDENT ASSURANCE TESTS
☐ DIST. LAB
☐ TRANS. LAB
☐ SPECIAL TESTS

SAMPLE SENT TO:
☒ HDQTRS. LAB
☐ BRANCH LAB
☐ DIST. LAB

FIELD NO.
DIST. LAB NO.
LOT NO. 1 - J11-11700
SHIPMENT NO.
P.O. OR REQ. NO.
AUTHORIZATION NO.

SAMPLE OF 4 EA #11 DEXTRA BARTEC REBAR COUPLERS
FOR USE IN BREA OH T-GIRDERS & TOP DECK REINFORCEMENT
SAMPLE FROM PRODUCTION LOT #1 (JOB # J11-11700) HEAT# NF11201061
DEPTH
LOCATION OF SOURCE DEXTRA AMERICA NUCOR - NE, USA.
THIS SAMPLE IS SHIPPED IN (NO. CONTAINERS) 1
AND IS ON E OF A GROUP OF 1 LOT
SAMPLES REPRESENTING (TONS, GALS, BBLS, STA. ETC.) 160 EA COUPLERS
OWNER OR MANUFACTURER DEXTRA AMERICA, INC.
TOTAL QUANTITY AVAILABLE
TEST RESULTS DESIRED ☐ NORMAL ☒ PRIORITY
DATE NEEDED 11-28-2011
REMARKS SEND TO TRANS LAB IN SACRAMENTO FOR QA TESTING - SERVICE SPLICE
CONTACT THEO @ 949.279.8412
THEO.PHAM@DOT.CA.GOV
COVER ADDITIONAL INFORMATION WITH LETTER
DATE SAMPLEED 11.23.2011
BY THEO PHAM TITLE S.R. / QA
DIST., CO., RTE., PM 12-ORA-57-18.4/20.9
LIMITS SR 57 YORBA LINDA TO LAMBERT
CONT. NO. 12-0F0324
FED. NO. NONE
RES. ENGR. OR SUPT. JAMES YU
ADDRESS 3020 SATURN ST., SUITE 200, BREA, CA
CONTRACTOR CC MYERS, INC.

MAIL TO SAME DESTINATION AS SAMPLE

Figure No. 2: Form TL-101, *Sample Identification Card*.

When completing Form TL-101, ensure that all items are completed. If incomplete, it could delay METS ability to issue test results. Specific items to consider are:

- Contact information for the person that did the sampling is needed to answer potential questions.
- Email or Fax number is used for METS to send the test results to expedite obtaining results.
- Include with the couplers, a copy of the Material Test Report (MTR) for the heat number of the bar reinforcing steel and the MTR for the lot number of couplers represented by the samples. METS cannot issue test results without this information.

Splice Acceptance Requirements

Slip Test Requirement

Except for mechanical lap, welded, or hoop splices, test one (1) of the four (4) splice test samples for total slip. If the slip test result complies with the total slip value requirement specified in the Standard Specifications[3], proceed to perform the tensile and/or rupture tests.
If the splice test sample exceeds the total slip value specified in the Standard Specifications[4], test the three (3) remaining test samples for total slip. If any of the three (3) remaining test samples exceed the specified total slip value, the Department rejects all splices in the lot.

Other Requirements for Service Splice

Service splices must develop a minimum tensile strength of 80,000 psi.
Acceptance:

- If only one (1) splice test sample complies with the requirements, the Department rejects all splices in the lot.
- If only two (2) splice test samples comply with the requirements, perform one (1) additional service splice test consisting of four new splice test samples on the same lot of splices. This additional test must consist of tensile testing four (4) splice test samples, randomly selected by the Engineer and removed from the lot of completed splices. If any of the four (4) splice test samples from this additional test do not attain the specified minimum tensile strength, the Department rejects all splices in the lot.
- If three (3) or more splice test samples comply with the requirements, the Department accepts all splices in the lot.

Other Requirements for Ultimate Butt Splice

Ultimate butt splices must do one of the following:

1. Rupture in the reinforcing bar outside of the affected zone and show visible necking as specified in California Test 670, Necking (Option I).
2. Rupture anywhere and neck as specified in California Test 670, Necking (Option II).

Acceptance:

- If only one (1) splice test sample complies with the requirements, the Department rejects all splices in the lot.
- If only two (2) of the four splice test samples comply with the requirements, perform one (1) additional ultimate butt splice test consisting of four new splice test samples on the same lot of splices. If any of these four (4) new splice test samples do not comply with the specified requirements, the Department rejects all splices in the lot.
- If three (3) or more splice test samples comply with the requirements, the Department accepts all splices in the lot.

Figure No. 3 depicts terms used in California Test 670.

[3] SS 2006, Section 52.1-08B(1) or SS 2010, Section 52-6.02B
[4] SS 2006, Section 52.1-08B(1) or SS 2010, Section 52-6.02B

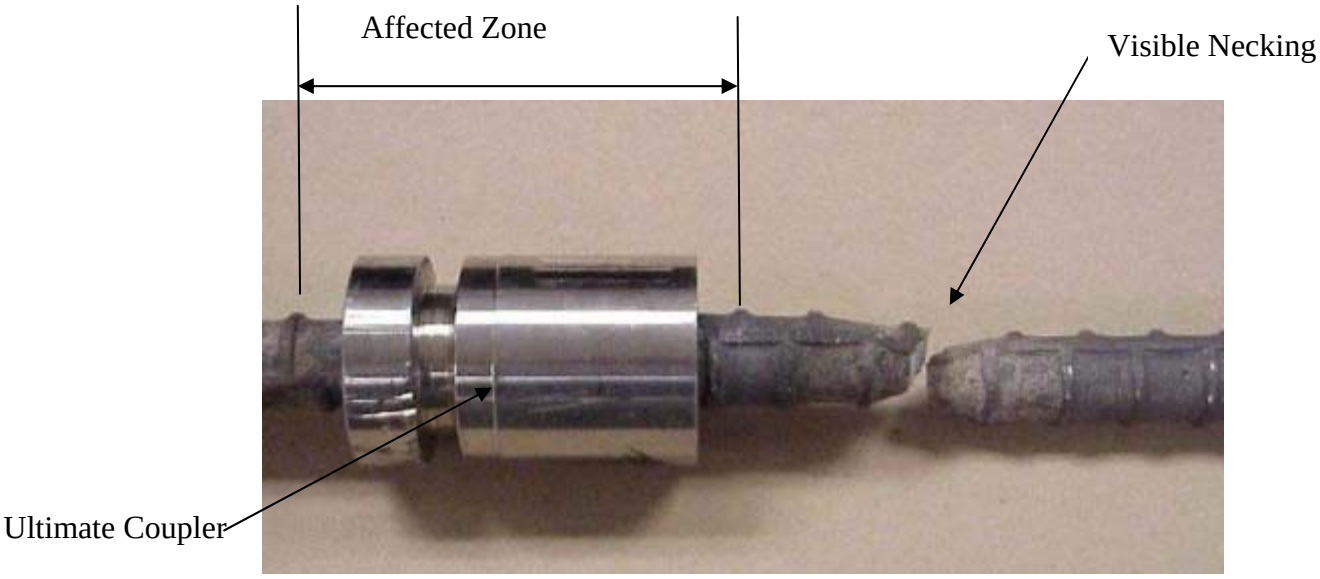


Figure No. 3: Passing Tensile Tested Ultimate Coupler. Note the bar rupture outside the affected zone and the visible signs of necking.

Mechanical Splice Acceptance Procedure Flow Chart

Figure No. 4 is a flow chart summarizing the Mechanical Splice Acceptance Procedure.

Mechanical Splice Acceptance Procedure Flow Chart

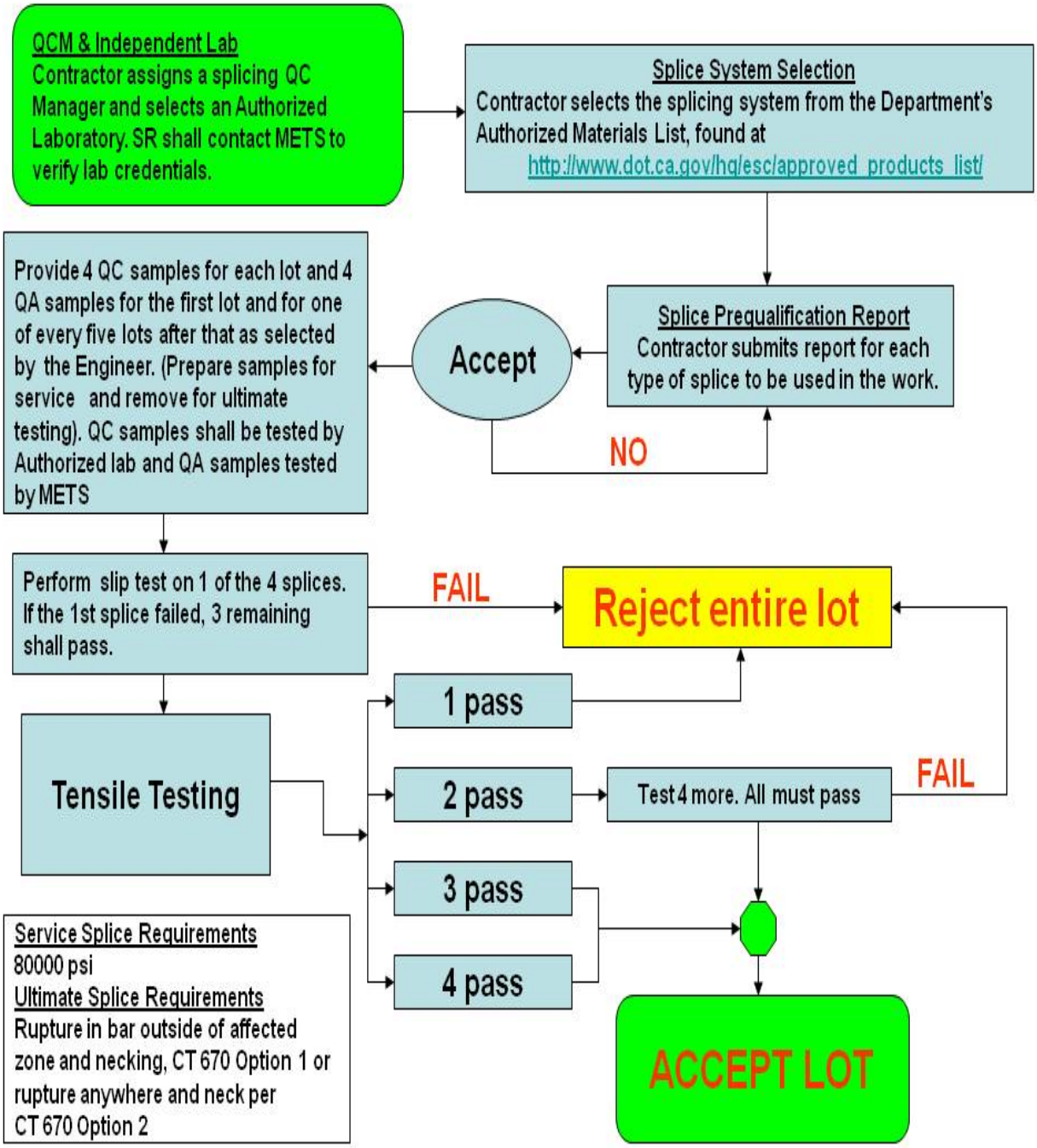


Figure No. 4: Mechanical Splice Acceptance Procedure Flow Chart.

Review Time

The specifications include a review time for production and quality assurance tests. To avoid costly delays, it is important to respond to the contractor in writing within the time specified in the specifications as shown in the table below.

Sample Type	Review Time
Production Sample Tests	Three business days to review each production test report submitted by the QCM.
Quality Assurance	Three business days upon receipt of the samples by METS. Two extra business days per each simultaneous submittal.

What To Do After Completion of Splice Production Work

Review Reports
The Contractor provides the following documents to the Engineer for review:

- Production Splice Report.
- Splice Rejection Report.
- Radiographic Film Developing Process Records (when welding is involved).

After completion and acceptance of the bar splices, ensure that the locations of the splices are noted in the As-Builts and that all the submittals and test reports are filed in the project records.

Testing Requirement Clarifications for Welded Butt Splices

To follow is information to clarify test requirements for welded butt splices.
1. Welded hoops:

- Slip test is not required.
- Destructive testing is required.
- Radiographic testing is not required.

2. CJP (Complete Joint Penetration) butt-welded splices (except welded hoops):

- Slip test is not required.
- Destructive testing is required for both service and ultimate butt splices.
- Radiographic testing is not required whenever butt-welded splices are removed from a lot of completed splices (i.e., whenever they require replacement).

- Radiographic testing is required whenever samples are prepared as described in the Standard Specifications[5] (i.e., whenever they do not require replacement due to removal from a completed lot).

3. Refer to California Test 670 regarding tensile test (destructive testing) requirements.

Items To Be Recorded In the Job Files

During the progress of the work all splice documentation shall be filed in Category 37 under the appropriate sub-category headings.

A list of items to be recorded in the job files are:

- Contractor's submitted plan designating the splicing QCM and testing laboratory.
- Laboratory Qualification (record of verification by the Structure Representative).
- Splice Prequalification Report.
- Test reports submitted by the splicing QCM.
- Summary record of production tests (see Figure No. 5 below).
- Test results of sample splices sent to METS.
- Records indicating the resolution of any failed QA test results.
- Certificates of compliance.
- As-Builts: Locate all reinforcing splices on the As-Built plans. See BCM 9-1.0.

[5] 2006 SS, Section 52-1.08C(3)(c) or SS 2010, Section 52-6.01D(4)(d)(ii).

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 165-11.0 REINFORCING STEEL**
Approved:_____	**Date: April 29, 2011**
ROBERT A. STOTT, Chief	**Expires: None**
Offices of Structure Construction	**Supersedes: None**

Subject: Allowing the Use of Plastic Spacers in Cast-In-Place Concrete Piles

Background

Plastic spacers are used to ensure concentric spacing for the entire reinforcing steel cage in Cast-In-Place (CIP) concrete piles. It allows travel of the cage along the wall of the drilled shaft excavation minimizing dislodging soils and accumulation of loose material in the bottom of the excavation. It provides for adequate clearance for fresh concrete to flow up the annular space between the cage and the side of the excavation as well as to maintain a minimum concrete cover. The main concern for disallowing plastic spacers for CIP piles is the difference in the coefficient of thermal expansion between the plastic and concrete.

In order to address this concern, the industry standards and the Concrete Reinforcing Steel Institute (CRSI) recommend that all plastic-side-form spacers should have at least 25% of their gross plane area perforated to compensate for the difference in the coefficient of thermal expansion between the plastic and concrete.

Current Practice

The 2006 Standard Specifications Section 52-1.07, "PLACING" does not allow the use of plastic supports for placement of reinforcement. There is no current practice to allow the use of plastic supports for placement of reinforcement.

New Practice

The new practice allows the use of plastic spacers in CIP piles provided the requirements of the forthcoming Standard Special Provisions are met.

The following new Standard Special Provision has been written to allow the use of plastic spacers:

Plastic spacers used in CIP piles between the outside of the pile bar reinforcing cage and the side of the drilled hole shall meet the following criteria:

- Spacers shall be used near the bottom, the top and at intervals not exceeding 10 feet vertically or per manufacturer's recommendation, whichever is less.
- The spacers shall be of sufficient size, composition and durability to support the lateral loads placed by the reinforcing cage upon the sidewall of the shaft. The spacers shall also be capable of withstanding impacts during the construction process.
- A minimum of 3 spacers shall be required at each level or per manufacturer's recommendation, whichever is greater.
- Spacers shall be placed a minimum of 5 inches clear of inspection tubes.
- The spacers shall be of adequate dimension to ensure an annular space of not less than 3 inches between the outside of the pile bar reinforcing cage and the side of the excavation along the entire length of the CIP pile.
- Plastic spacers shall have at least 25% of their gross plane area perforated to compensate for the difference in the coefficient of thermal expansion between the plastic and concrete.
- Plastic spacers shall conform to the provisions in Section 3.4. "All-Plastic Bar Supports" and Section 3.5. "Side-Form Spacers" of the Concrete Reinforcing Steel Institute Manual of Standard Practice (See Attachment No. 1).
- Plastic spacers shall be commercially manufactured. (See Attachment No. 2 for example of commercially manufactured plastic spacer.)
- The Contractor shall submit to the Engineer for review and approval the manufacturer's data and a sample of the proposed plastic spacer.

Future advertised projects will include this new specification.

Attachment No. 1*

Sections 3.4. and 3.5. of the CRSI, Manual of Standard Practice, 28th Edition

3.4. All-Plastic Bar Supports

The industry practices presented in this section are intended to serve as a guide for the selection and utilization of all-plastic bar supports used to position reinforcing bars in reinforced concrete.

All-plastic bar supports may have a snap-on action or other method of attachment. All-plastic supports are lightweight, non-porous and chemically inert in concrete. Properly designed all-plastic bar supports should have rounded seating to avoid punching holes in the formwork and should not deform under load when subjected to normal temperatures encountered in use nor should they shatter or severely crack under impact loading when used in cold weather.

All-plastic bar supports will not rust, therefore eliminating blemishes on the surface of the concrete. These supports are particularly suitable in situations of moderate to severe exposure or when grinding of the concrete is necessary. All-plastic bar supports may be used to support epoxy-coated reinforcing bars (see Section 3.6). These bar supports provide maximum rust protection, i.e., Class 1.

3.5. Side-Form Spacers

A side-form spacer is a type of bar support which is used to maintain side concrete cover on the reinforcing bars against a vertical form, such as for walls and columns. Spacers can also be used to align a reinforcing bar cage in a drilled shaft. Spacers can be made of steel wire, precast concrete, or plastic. Examples of side-form spacers are SBC wire bar supports, DSSS and DSWS precast concrete bar supports, and WS, DSWS and VLWS all-plastic bar supports.

Typically these supports are not shown on the design or contract drawings. There are numerous variations in the Placer's requirements making it difficult for the supplier to estimate. Estimating, detailing or furnishing these materials is not a normal industry practice unless by special arrangement with the General Contractor or Placer. Unless agreed to between the Buyer and the Seller, these materials will not normally be included in the Supplier's bid.

* Published with permission of the Concrete Reinforcing Steel Institute

Attachment No. 2

Sample of Commercially Manufactured Plastic Spacer.

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MEMO NO.*	ISSUE DATE	TITLE
168-1.0	11/15/77	Bolted Connections for Overhead Sign Structures
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168-3.0	08/05/87	Shop Plans for Signs
168-4.0	07/19/12	Resin Capsule Anchorage for Bridge Mounted Signs

Robert A. Stott, Deputy Division Chief
Offices of Structure Construction

**Denotes the document is a Bridge Construction Bulletin*

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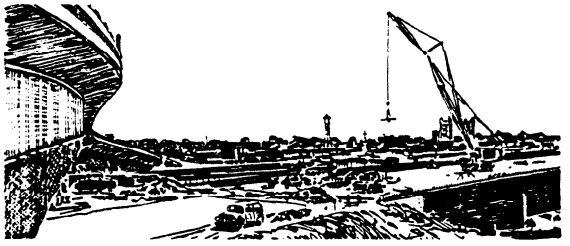

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BOLTED CONNECTIONS FOR OVERHEAD SIGN STRUCTURES

Following is a summary of information which will aid personnel charged with the responsibility of inspecting bolted connections for overhead sign structures.

Unless otherwise specified, all bolts and nuts shall conform to the specifications of ASTM Designation A-307. Also, unless otherwise specified, A-307 bolts should be furnished with commercial quality washers, have hex heads and nuts, and should be "wrench tight". A-307 bolts should be of such length that they extend entirely through the nut (or nuts), but not more than ¼ inch beyond. A-307 bolts in shear shall have not more than one thread within the grip.

Anchor bolts for sign foundations shall conform to the specifications of ASTM Designation A-307. These bolts, washers, and nuts should be galvanized as specified. In addition, the nuts should be retapped after galvanizing to avoid galling and thread stripping. Anchor bolts with properly retapped nuts and clean threads should be tightened sufficiently to prevent removal by hand (wrench tight).

Where high-strength bolts are specified for overhead sign structures, the bolts, nuts and washers shall conform to the specifications of ASTM Designation A-325. ASTM A-325 requires that the bolt head be marked "A-325". In addition, the bolt head may be marked with 3 radial lines spaced 120 degrees apart. High-strength nuts will be marked with the number "2" or "2H", by three equally spaced circumferential lines, or by the letters "D" or "DH".

High-strength bolts used in overhead sign structures may be tightened by any method in order to obtain the required tension.

As indicated in STANDARD SPECIFICATIONS SECTION 55-3.14, a (flat) hardened washer must be installed under the high-strength nut or bolt head, whichever will be turned to tension the bolt, regardless of the method used to tension the bolt, or the type of connection design. Lock washers are not an allowable substitute.

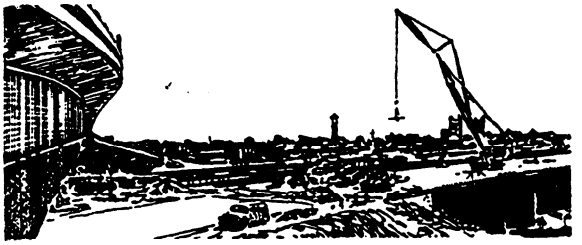
For overhead sign structures, measurement of the bolt tension of field connections shall be by approved direct tension indicators furnished by the contractor. Assembly of high-strength bolted connections for sign structures may be performed with galvanizing or paint on the contact surfaces.

If raised-lug washer-type direct tension indicators are used, one indicator shall be furnished and installed with each bolt in accordance with the following:

Non-galvanized indicators shall be cleaned in a compatible solvent, dried, and dipped in a 1:l solution of Zinc-Rich Primer, Section 91-2.01 and thinner specified therein. Indicators shall be allowed to dry before use.

Washer-type tension indicators shall be installed so that the lugs bear against either a hardened washer or an element of the fastener which is not turned during tightening. After snugging up all bolts of the joint, tightening shall progress from the most rigid part of the joint to the free edges. Bolts shall be tightened until the average gap around the perimeter of the washer is equal to 0.007-inch. Gaps shall not be completely closed. Washer-type indicators shall not be reused,

In connection with tensioning high-strength bolts, the threads of nuts and bolts should be properly prepared to prevent "galling" and excessive friction losses. Therefore, nuts and bolts which are not galvanized ("black") should be clean and dry or lightly oiled. Nuts for high-strength galvanized bolts should be overtapped after galvanizing, and then treated with a lubricant that is clean and dry to the touch. No attempt should be made to tension a high-strength bolt with an un-retapped galvanized nut. The bolt threads will usually gall and strip before the required bolt tension is reached.



EXPANSION ANCHORS FOR BRIDGE MOUNTED SIGNS

Threaded stud bolts must be used when using expansion anchors to attach bridge mounted signs on concrete structures. Headed bolts must not be used in place of threaded stud bolts.

The reason for this is that the use of the headed bolt gives questionable results. as there is no way of confirming whether the expansion anchor has been firmly seated in the concrete, or if it has merely cinched up against the mounting bracket.

For additional information relative to expansion anchors, and for a list of expansion anchors that have been approved by the Transportation Laboratory refer to Bridge Construction Memo 135-5.0.

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SHOP PLANS FOR SIGNS

On projects specifying shop plan submittals for signs, plan approval is the responsibility of the Resident Engineer. The Office of Structure Design (OSD) will not review the plans unless unusual details or circumstances exist. Under those conditions, OSD will provide a cursory review upon request to evaluate structural concerns and substantiate design intent. However, final responsibility rests with the Resident Engineer. TRANSLAB should be consulted regarding material questions.

In contracts with "Buy America" clauses, fabricators frequently have difficulty supplying some of the special American steels specified because of the small quantities involved. Some of these steels are more easily obtained from foreign sources and can be used if the quantity or cost does not exceed that allowed by the contract. To comply with the "Buy America" provisions substitutions of steel type may be made by change order if TRANSLAB verifies equivalent structural and galvanizing properties.

Subject: Resin Capsule Anchorage for Bridge Mounted Sign Structures

Background

The State Bridge Engineer policy memo dated July 21, 2008, *Policy for Attachment of New Bridge Mounted Signs to Soffit Slab, Underside of Bridge Decks or Deck Overhangs*[1] states:

> "Effective immediately, all new bridge mounted signs, including Changeable Message Signs, that are suspended from the bridge soffit, underside of bridge decks or deck overhangs by anchorage into concrete shall be anchored using either cast-in-place anchors or through bolts as appropriate. …resin capsules shall not be used to anchor such sign structures."

The intent of this policy memo was to prohibit the use of resin capsule anchorage (RCA) in direct, sustained tension applications because of their susceptibility to creep. The picture below provides an example of what is considered direct, sustained tension where the use of a RCA is prohibited. The prohibition in the policy memo does not apply to smaller barrier rail mounted sign structures.

This BCM is intended to address recurring questions that have been received by the *DES Signs and Overhead Structure Specialist* related to RCA for bridge mounted sign structures since the July 2008 policy memo was written.

[1] See Attachment No.1: Policy Memo dated July 21, 2008, Subject: *Policy for Attachment of New Bridge Mounted Signs to Soffit Slab, Underside of Bridge Decks or Deck Overhang*.

Guidelines

Bridge mounted signs on Caltrans' projects typically conform to the two mounting arrangements that are illustrated in the Figure No. 1 and Figure No. 2. They are:

1) RCAs placed into the barrier rail (location A) and into the edge of deck (location B), and into the exterior girder or the underside of the deck overhang (location C). This type of a support system is typically used for bridge mounted signs which are over 100 inches tall. See Figure No. 1.

2) RCAs placed into the edge of deck (location B), and into the exterior girder or the underside of the deck overhang (location C). See Figure No. 2.

Mounting signs consistent with one of these configurations is consistent with the 2008 policy memo. If properly installed, the RCAs will perform as designed with minimal creep that will not impact their capacity.

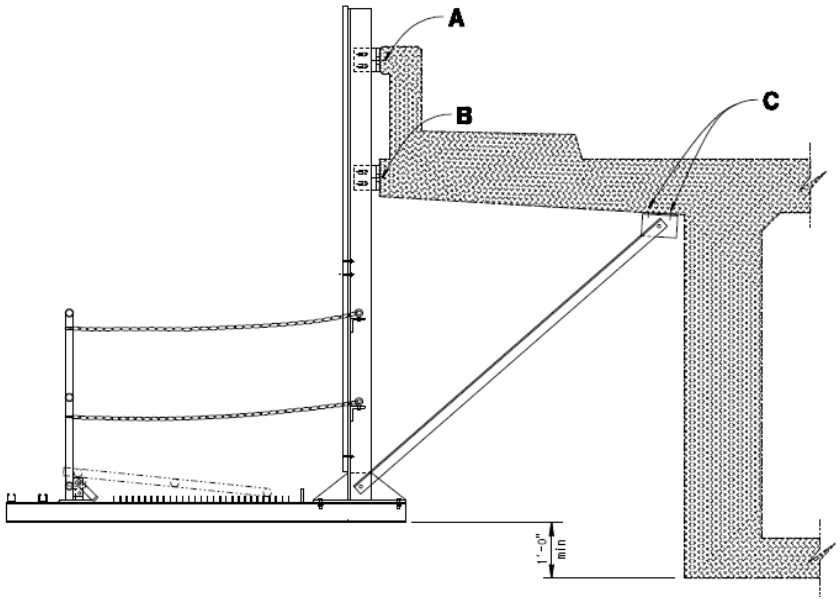

Figure No. 1

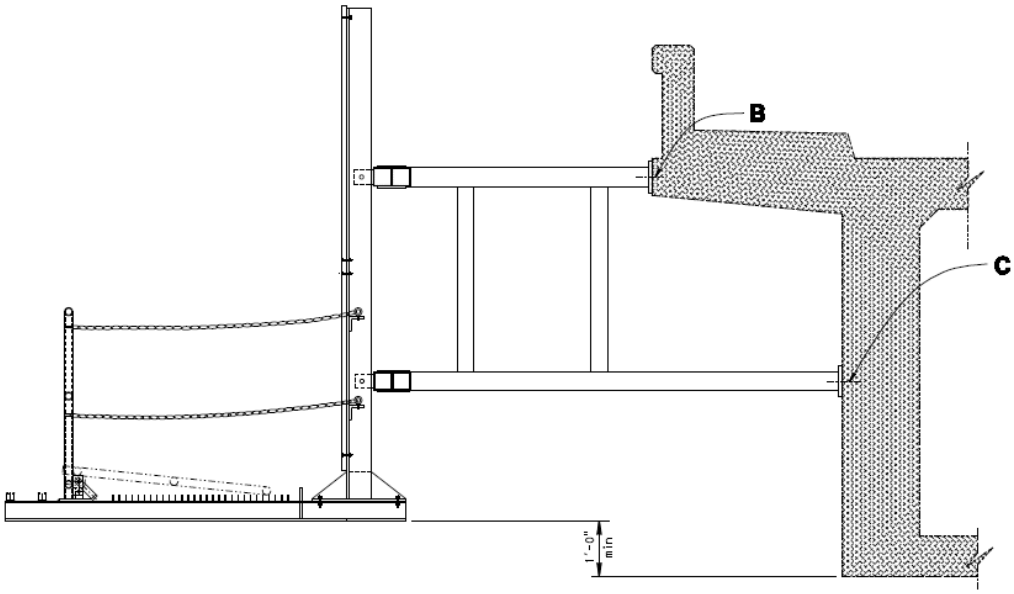

Figure No. 2

Structure Representative Responsibilities:

- If details deviate from the mounting configurations described above, contact the *DES Signs and Overhead Structure Specialist*.
- Verify that the RCA is on the *Authorized Materials List*[2] at: http://www.dot.ca.gov/hq/esc/approved_products_list/pdf/resin_capsules.pdf
- Verify that the proposed system does not interfere with any other embedment or risk damaging the prestressing system.
- Notify the contractor of their authorization for use of the selected RCA.
- Verify that RCAs are installed in accordance with the manufacturer's recommendations, including using the manufacturer recommended drilling equipment.
- Cored holes are not permitted under any conditions.
- Verify that the holes are clean and dry.
- Do not install RCAs in the rain.
- Verify the size of the hole (diameter and depth).
- Verify that the required embedment depth and minimum edge distances are obtained.
- Ensure that the RCAs are loaded only after the adhesive *cure time* has elapsed (shown in the *Authorized Materials List*).

[2] Equivalent terms that may have been used in other documents that have the same meaning would include; *Approved Materials List*, *Pre-Qualified Products List*, or *Approved Product List*.

State of California
DEPARTMENT OF TRANSPORTATION

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: DES DEPUTY DIVISION CHIEFS

Date: July 21, 2008

TERRY L. ABBOTT
Chief
Division of Design

MARK LEJA
Chief
Division of Construction

BARTON NEWTON
Assistant Division Chief
Structures Maintenance and Investigations
Division of Maintenance

From: KEVIN J. THOMPSON
State Bridge Engineer
Division of Engineering Services

Subject: Policy for Attachment of New Bridge Mounted Signs to Soffit Slab, Underside of Bridge Decks or Deck Overhangs

Effective immediately, all new bridge mounted signs, including Changeable Message Signs, that are suspended from the bridge soffit, underside of bridge decks or deck overhangs by anchorage into concrete shall be anchored using either cast-in-place anchors or through bolts as appropriate. Cartridge epoxies, chemical adhesives or resin capsules shall not be used to anchor such sign structures.

For the types of bridge mounted signs identified in the preceding paragraph that are in construction, a Contract Change Order shall be issued showing anchorage to the bridge using either cast-in-place anchors or through bolts.

This policy has been developed to increase the structural safety of sign structures that are suspended from bridges.

Contact Madhwesh Raghavendrachar, Senior Bridge Engineer, Structure Design, Division of Engineering Services at 916-227-7116 or mraghave@dot.ca.gov if there are any questions.

c: SD Office Chiefs
Susan Hida

"Caltrans improves mobility across California"

Volume II

BRIDGE CONSTRUCTION MEMO 170-0.0

STRUCTURAL STEEL

July 1, 2001

Sheet 1 of 1

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170-1.0	7-1-01	DISCUSSION OF BASIC CONSTRUCTION TERMS AND TOPICS FOR HIGH-STRENGTH BOLTED CONNECTIONS
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170-3.0	7-1-01	APPROVED METHODS OF TENSIONING HIGH-STRENGTH BOLTED CONNECTIONS
170-4.0	7-1-01	STRUCTURAL STEEL WORKING DRAWINGS

DOLORES M. VALLS, Deputy Division Chief
Offices of Structure Construction

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Volume II

BRIDGE CONSTRUCTION MEMO 170-1.0

STRUCTURAL STEEL

July 1, 2001

DISCUSSION OF BASIC CONSTRUCTION TERMS AND TOPICS FOR HIGH-STRENGTH BOLTED CONNECTIONS

Discussion of Structural Bolts

There are many types of bolts used for structural applications. Quality for all of these is ensured through compliance with specified American Society for Testing and Materials (ASTM) standard specifications. These national specifications clearly denote specific mechanical properties, chemical composition, and dimensions for each type of fastener.

Where lower strength fasteners are required, ASTM A307, mild steel fasteners and anchor bolts are commonly used. These are usually not preloaded, have a minimum yield strength of 36 ksi, are extremely ductile, and can be welded (when S1 supplementary requirements are specified) and zinc coated.

The main type of high-strength structural bolt frequently specified in Caltrans contracts for steel joints in bridges, overhead sign support structures, and buildings, is designated as ASTM A325, and is available only in a heavy hex headed style and in diameters from ½" through 1-1/2". A325 bolts are almost always specified for major structures, and have a minimum tensile strength of either 105 or 120 ksi, depending on the bolt diameter; the minimum proof load is either 81 or 92 ksi. Because Caltrans wants to insure maximum plastic ductility of fasteners in structural joints in the event of a large earthquake, we specify A 325 fasteners or F1852 tension control (TC) bolts almost exclusively. These can be zinc coated to insure a long life in corrosive coastal environments. While an A490 structural bolt is available, it's lower ductility and inability to be zinc coated make it less desirable for use in coastal regions where long-term corrosion protection is vital and earthquakes are likely to occur.

Where larger sizes of high-strength fasteners or threaded rods having properties identical to those of A325 bolts are required, an A449 series of bolt and rod is readily available. Mechanical properties and chemical composition of this fastener are identical to those of A325 bolts; it is available in a wider variety of sizes, from ¼" to 3" diameters, can be ordered in a number of different head styles, and can be zinc coated.

Another type of high-strength bolt and threaded rod which is quenched and tempered alloy steel, and is called an ASTM A354 is also readily available; it comes in two grades - BC and BD and in diameters from ¼" to 4". Because the tensile strength of Grade BD fasteners may exceed 150 ksi, they cannot be zinc coated. These two grades of fasteners are frequently used for large bolts or rods, where high strengths are required.

Discussion of Various Topics Related to High-Strength Bolting

In the following paragraphs, various topics related to high-strength bolting are discussed:

Types of Connections:
A bolted connection may be designed as either a ***bearing type*** or a ***slip critical*** connection. Caltrans Standard Specifications require that all connections made with high-strength bolts shall be considered as (slip-critical) ***friction-type*** joints, and shall be tensioned as a typical slip-critical joint, unless otherwise designated on the contract plans or specifications. To insure that adequate friction is developed between joint plies, faying (contact) surfaces of all high-strength bolted connections shall be free of rust, mill scale, dirt, grease or any other material foreign to the steel, before assembly. Specifications may require faying surfaces of bolted connections to be coated with either hot-dip zinc coating that has been hand wire brushed prior to assembly, or with an approved inorganic zinc primer prior to assembly.

Bolt Holes:
Bolt holes shall be either punched full size, drilled full size, sub-punched and reamed, or sub-drilled and reamed. Flame cutting of holes is not permitted. Reference Section 55-3.14A, "Bolt Holes" of the Caltrans Standard Specifications, and Table 1 in Section 3(c) of the RCSC Specification. For high-strength bolts, the diameter of standard bolt holes is 1/16" larger than the nominal diameter of the bolt shank.

Thread Stickout:
Determining and purchasing the correct bolt lengths for each different joint is the responsibility of the contractor. The amount of exposed thread beyond the outer face of the nut is called "thread stickout". After high-strength bolts have been installed and tensioned, the permissible range of thread stickout permitted is from flush to not more than 1/4 inch beyond the outer face of the nut. Note: On TC bolts frequently there are a few partial threads adjacent to the groove where the splined tail breaks off. Therefore for TC bolts, thread stickout shall be measured from the outer face of the nut to the first full thread near the sheared end of the bolt (after the splined end has been sheared off).

Hardened Washers:
According to Section 55-3.14 of the Standard Specifications, one (flat) hardened washer (ASTM F436 or F436M) must be installed under the nut or bolt head, whichever is the element turned in tightening. A maximum of one additional hardened washer may be installed under the non-turning element of the fastener assembly to correct excessive thread stickout. Regardless of the method used to tension the bolt, or the type of connection design, lock washers are not an allowable substitute for hardened washers. Lock washers generally do not have adequate surface contact area, or sufficient corrosion resistance, and due to different steel chemistry and thinner protective coatings, corrode at a higher rate than adjacent steels. If the slope of the exterior face(s) of the connected parts exceeds 1:20 (approximately 3 degrees) relative to the bolt or nut face, a hardened beveled washer(s) meeting requirements in ASTM Specification F436 shall be inserted against each sloped surface.

Snug-Tight Condition:
No matter which of the approved tightening methods is used to tension high-strength bolts, the first step in tightening a joint is the same - bring all plies in the joint in contact by snugging the fasteners. This requires all fasteners in a joint to be brought to a snug-tight condition using a systematic tightening sequence (starting from the center of the joint). "Snug-tight" is defined as the full effort of a person using a spud wrench or a few impacts of a pneumatic wrench applied to the nut. While snugging fasteners, if plies are not initially in contact, care should be taken to avoid bending of the connection parts. Following snugging, all plies in a joint must be in firm contact with each other.

Systematic Tensioning Pattern:
All bolts in a joint need to be tensioned in a systematic manner to produce a consistent even tension in each bolt. The tensioning pattern may be done in a crisscross or alternating fashion, and needs to be systematic to produce an even tension in all bolts. This tightening pattern should be used to bring bolts to the snug condition, and also to their final minimum required tension. In joints having a rectangular or square bolt pattern, bolts must be tensioned, starting at the center (most rigid part) of the joint and proceeding toward the free edges. For joints having a circular bolt pattern, a crisscross alternating pattern is appropriate. Writing a sequential number on each fastener in a large joint is a good way to insure all bolts are tensioned in their correct order, and none miss their turn. To insure that all fasteners are fully tensioned, this final tightening process may require more than one cycle.

Fastener Storage and Handling:
Storage: Regardless which of the approved methods is chosen to tension high-strength bolts, the condition of the fastener components (especially threads on both the nuts and bolts) is critical; all fastener components must be furnished and maintained in good condition until installed and final inspection has been performed. The original lubricant on all fastener components must be kept intact as supplied from the manufacturer, and all fastener components must be stored so that they do not get rusty or dirty. As soon as fastener containers are received at the job site, they must be stored in the original containers and protected from dirt and moisture. Containers should always be covered and be kept off the ground.

Handling: Fastener components from different lots must never be inter-mixed. Only those fastener components that are to be used in one shift are allowed to be removed from containers. Components not used during that shift must be returned to their original containers. The following information must appear on the outside of the shipping/storage containers:

1. Manufacturer's name and address.
2. Contents (size and numbers).
3. Component lot number.
4. Rotational capacity lot number.

Note: All components of galvanized fastener systems (including bolts, nuts, washers, and DTIs) must be shipped and kept together as an assembly.

Lubrication:
Plain (black) Fasteners: Most plain or "black" fastener components have been heat-treated and all parts are coated by the manufacturer with a thin film of water-soluble (oily) lubricant that can be easily washed-off if exposed to moist elements. Prior to being installed, threads on bolts and nuts shall be oily to the touch, as received by the manufacturer. Should the bolts, nuts, or washers show signs of improper storage, such as rust and dirt accumulation, or absence of original lubricant on the threaded fastener components, this shall be cause for rejection.

Zinc-coated Fasteners: All zinc-coated nuts used on high-strength zinc-coated bolts must be coated by the manufacturer with a lubricant that is clean and dry to the touch, unlike black bolts that are furnished in an oily condition. To make identification easier, a colored dye, or an ultraviolet dye that can be seen with a black light, is required in the lubricant used for all galvanized nuts. No attempt should be made to tension a high-strength, zinc coated bolt whose nut has not been lubricated with a dry lubricant or properly "tapped" oversize. Without the proper lubrication applied on the nut threads and base, the fastener threads can gall, strip or seize, causing the bolt to shear off before the required bolt tension is reached.

Rotational Capacity (RoCap) Test:
At the job site, a rotational capacity test must be done on each lot of both plain and galvanized fasteners to confirm that the nut lubricant, and thread fit and condition as received from the manufacturer will result in proper tensioning without galling or stripping of threads or shearing of the bolt and that the bolt has good ductility. The quality and amount of lubricant and thread fit and condition can vary considerably between various manufacturers and fastener lots, therefore, the use of torque values obtained from charts or tables, or by testing other lots of fasteners is not allowed.

Reuse of High-Strength Fasteners:
Black A325 nuts and bolts may be reused once if allowed by the Engineer. However, neither A490 fasteners nor galvanized A325 fasteners shall be reused after they have been tensioned. Reuse of black A325 bolts and nuts should only be considered if they are in good condition (clean and with lubricant), the bolt threads are not excessively elongated (checked by spinning the nut by hand over the entire length of bolt threads), and each fastener lot is retested and passes the new pre-installation and rotational capacity tests. Once installed, neither TC bolt assemblies nor direct tension indicators (DTIs) may be reused.

Inspecting a Completed Bolted Joint:
Section 55-3.14, "Bolted Connections" of the Caltrans Standard Specifications states, "Bolt tension shall be verified by applying a job inspecting torque to nuts at locations selected by the Engineer. Inspection of each joint should be done as soon as possible, just after tensioning of all fasteners in a joint has been completed. At least 10% of the fasteners in each joint shall be checked. Verification of bolt tension shall be done by the Contractor in the presence of the Engineer and in such a manner that the Engineer can read the torque wrench gage or see gaps around the DTI during checking." The job inspecting torque shall first be determined by the Contractor by testing five fasteners from each lot of bolts according to the procedure detailed in Section 9 (b)(3) of the RCSC Specification. To verify adequate tension in each of the fasteners selected for

inspection in a completed joint, a suitable manual torque wrench (dial or digital read out only) is used to apply the job inspecting torque value to nuts (or bolt head, if turned). During the inspection, if any of the nuts turn, then 100% of the bolts in the connection shall be tested, and all bolts found to be under tensioned shall be tightened, and then reinspected.

Definition of Terms Commonly used in High-Strength Bolting

Term's commonly used in high-strength bolting operations and specialized tools need to be clearly understood. The following is a list of terms and tools that are frequently used when dealing with high-strength bolts. Inspectors and construction personnel need to be familiar with these - what they are and how to use them. They include:

Bolt tension calibrator: A machine to measure bolt tensions (i.e., Skidmore-Wilhelm, or Norbar).

DTI (direct tension indicator): A device installed on high-strength bolts to monitor bolt tension. It must conform to requirements in ASTM F959/F959M.

Typical direct tension indicator (DTI)

Electric installation tool for tension control (TC) bolts: An electric tool used to install TC bolts.

Faying surfaces: Contact surfaces between structural plates within a high-strength bolted joint.

Grip length: The total thickness of all plies in a joint, including washers (distance between the underside of the bolt head and the inside face of the nut.

Pre-installation testing: A test series performed on each lot of fasteners, and at the beginning of a shift or job in which the installer demonstrates that with the actual installation equipment and lot of fasteners to be used on the structure, he can properly install them and obtain the proper tension.

Job inspecting torque: A torque value established for each lot of fasteners, and used after a joint has been completed to check that bolts have been tightened to at least the minimum tension.

Match marking: A series of four marks made on the outer surface of a joint, after all fasteners in a joint have been snug tightened to monitor the amount the nut has been turned. Match marking is required if the turn-of-nut tensioning method is used.

Mechanical deposited and hot-dip zinc coating: Two different coating processes where zinc metal is applied to surfaces of fastener components.

Rotational capacity (RoCap) test: A preliminary test performed both by the manufacturer and at the job site on new fasteners to insure that there is proper lubrication on fastener threads and that there is adequate ductility.

Snug tight: The preliminary tightening stage that all fasteners in a joint must be taken to, that produces a tension in each fastener of about 10% of its final tension, and that brings all plies of a joint into firm contact.

Tension Control (TC) fastener: An alternative high-strength fastener system, which includes a nut, washer, and bolt with a splined end. It must conform to requirements in ASTM Specification F1852.

Typical twist-off type TC fastener system

Thread stickout: Amount of threaded bolt tail projecting beyond the outer face of the nut on an installed bolt.

Torque multiplier: A tool used to amplify tightening effort applied to tension (install) or inspect large high-strength bolts.

Torque wrench: A tool (dial or digital type permitted) used to tighten and inspect high-strength bolts.

Volume II

INSPECTION PROCEDURE FOR CHECKING TENSION IN HIGH-STRENGTH BOLTS

Introduction

Following is a brief summary of information that will aid personnel charged with the responsibility of inspecting high-strength bolted connections.

Phases of Inspection

There are three main phases of inspection necessary when high-strength fasteners are installed. These are: 1) Preliminary inspection and testing, 2) Inspection during high-strength fastener installation, and 3) Inspection after high-strength fasteners have been installed.

Phase 1 - Preliminary Inspection and Testing

1. Sampling components and laboratory quality assurance testing:
Fasteners arriving at the job site should be sampled and tested by Caltrans to insure compliance to American Society for Testing and Materials (ASTM) requirements prior to use.

2. Pre-installation testing:
After the satisfactory quality of fasteners is confirmed, the contractor is required to perform pre-installation testing. A calibrated bolt tension-measuring device (Skidmore-Wilhelm or Norbar) is required for this testing. This testing will demonstrate that the contractor has proper equipment and knowledgeable personnel to correctly install high-strength fastener systems being used and can obtain the proper fastener pre-tension for all lots of fasteners to be used. This includes insuring that "snug- tight" tension is correct, impact wrenches and torque wrenches produce the adequate minimum tension, the correct size of calibrated wrench is used (it should take about 10 seconds to fully tension a fastener with a pneumatic or hydraulic wrench).

3. Rotational capacity (RoCap) testing:
This test will verify that the quantity and quality of lubricant and numerous other variables affecting nut factors including thread fit and condition and coating type and thickness will allow fasteners to be tensioned without galling or stripping.

When doing RoCap testing for all lots of fastener systems, a calibrated bolt tension measuring device (calibrated within the last year and traceable to the National Institute of Standards and Technology) shall be used. If fasteners are too short to fit in a bolt tension meter and obtain a full nut, then the short bolt test procedure, as outlined in the current Caltrans Standard Special Provisions shall be used.

Phase 2 - Inspection during High-Strength Fastener Installation
The Inspector shall verify that:

1. The contractor has chosen an acceptable type of high-strength fastener systems as permitted in the contract. Acceptable types may include:
    A. Black bolt (ASTM A325) [with a suitable nut (ASTM A563) and washer (ASTM F436)].
    B. Zinc-coated bolt (ASTM A325) [with a suitable nut (ASTM A563) and washer (ASTM F436)].
    C. Tension control (TC) fastener assembly (ASTM F1852).
    D. Black or mechanically zinc-coated bolt (ASTM A325) [with a zinc-coated Type 325 DTI (ASTM F959), suitable nut (ASTM A563) and washer (ASTM F436)].

2. The contractor is using an approved method of installing high-strength bolts and maintains proper installation technique throughout the project. Approved installation methods include:
    A. Turn-of-nut.
    B. Calibrated wrench [impact wrench (pneumatic, hydraulic, or electric) with positive shut-off system or manual torque wrench - dial or digital only]
    C. Direct tension indicators (DTI's) with black or mechanically zinc-coated bolts.
    D. Tension control (TC) fastener assemblies.

3. All high-strength bolts are installed with a flat hardened washer under the nut or bolt head, whichever is the element turned in tightening. A maximum of one additional hardened washer may be installed under the non-turning element of the fastener assembly so as to prevent the nut from "bottoming out" within the thread transition zone on the bolt shank. (Lock washers are not an allowable substitute).

4. A back-up wrench is used on each fastener to prevent the non-turning element (usually the bolt head) from turning while the fastener is being tensioned.

5. Installation tests have already been run for all equipment and workers involved, and for each different lot of fasteners used. If a different lot of fasteners or installation equipment is used, or new or different installation crewmembers begin work, new pre-installation tests must be conducted.

6. All fasteners in a joint are installed and tensioned at one time. (It is not acceptable to partially install some of the bolts in a joint, or to "stuff" bolts in a joint and let them remain loose for long periods untensioned)

7. All fasteners, no matter which type are used, shall first be taken to a "snug-tight" condition in a systematic tightening pattern, and then fully tensioned in stages using a systematic tightening pattern.

8. Faying surfaces of all plies in each joint and are in firm contact with each other after the members have been brought to a "snug-tight" condition (defined as the full effort of a person using a spud wrench or 12" flex-handle and socket).

9. No short cuts are taken in the proper installation procedure.

10. The fasteners are properly stored after each shift is done and are not allowed to be exposed to degrading elements (especially rain, fog, dampness, dirt, wind, or extreme temperatures).

Phase 3 - Inspection after High-Strength Fasteners Have Been Installed

After all fasteners have been installed and fully tensioned, a final inspection check is done to ensure the job was done properly. This includes 1) a visual check to confirm all plies of a joint are in firm contact, especially around bolts, 2) a check of tension in 10% of the fasteners in each connection (but not less than two) using a torque wrench (dial or digital gage) to confirm that minimum required bolt tension has been attained. This torque requires that a “job inspecting torque” be determined by the contractor for each different lot of fasteners used. A bolt tension calibrator should be used to establish the "job inspecting torque”. Bolt tensions in a joint should be inspected immediately after a joint has been completed. If nuts on any of the bolts checked during the inspection move prior to reaching the job inspecting torque, the remainder of the fasteners in the connection should be inspected and retensioned. Directions for establishing a job inspecting torque value and adjusting tensions in loose bolts are found in paragraph 9(c) of the RCSC Specification (Reference 4 of Attachment No. 3) and shall be followed. Methods for inspecting short bolts are contained in the Structural Bolting Handbook [SBH] (Reference 10 of Attachment No. 3) and require the use of DTIs. Joint seams shall be caulked if needed after fastener tensions in the connection have been inspected and the joint has been approved.

Besides checking bolt tension, the thread stickout should be checked to verify that it is between 0 (flush) and 1/4” beyond the outer face of the nut and that it is the same for all fasteners of similar length. An equal amount of thread stickout in each bolt is an indication that bolt tensions are consistent. Variations in bolt stickout are an indication that some fasteners may be undertensioned, or that joint plies are not in firm contact. Additionally, variations in the thread stickout could indicate that fasteners from different lots have been improperly utilized within the same joint.

It is the contractor's responsibility to provide all required testing equipment and to perform the tests in the presence of the Engineer. If needed, the Division of Structure Construction has bolt tension calibrators and torque wrenches that are available for use by Caltrans personnel for quality assurance inspection.

Attachments No. 1 contain answers to frequently asked questions regarding high-strength fasteners. Attachment No. 2 is a list of specifications and references for high-strength bolting.

COMMON QUESTIONS AND ANSWERS CONCERNING HIGH-STRENGTH FASTENERS

1. Q. What is a Pre-Installation Test (also called an Installation Verification (IV) or Calibration Test)?
   A. The pre-installation tests are performed by the Contractor's personnel using the same installation equipment and witnessed by the Engineer. At least three fasteners from each lot shall be tested in a bolt tension calibration device; if bolts are too short to be installed in such a device, then DTIs and the procedure outlined in the SBH (Reference No. 10 of Attachment No. 3) shall be followed. Rules and required testing frequency are described in Section 8(d) of the RCSC Specification (Reference No. 4 Attachment No. 3). These pre-installation tests will determine the ability of the Contractor's personnel, equipment and procedures used in the actual construction to properly install the same high-strength fasteners used in the structure, according to the approved installation method specified or chosen.

2. Q. What is a Rotational Capacity (RoCap) Test?
   A. This test must be performed by the manufacturer/supplier according to the procedure in the Caltrans Standard Special Provisions. The Contractor is also required to perform the RoCap test at the job site using the same test procedure. This test will verify that the various lots of fastener assemblies when finally ready to be installed at the job site, are capable of withstanding a prescribed nut rotation without failure of the fastener (insures good ductility of fastener), that nuts have been properly lubricated in order to prevent seizing or galling of the threads, and that bolts and nuts are properly tapped and heat treated to prevent thread stripping.

3. Q. Do RoCap tests need to be done on TC bolts, and on fasteners on which DTIs have been installed?
   A. Yes.

4. Q. How many bolt assemblies are necessary for each test required?
   A. Pre-Installation Test: 3 minimum per lot (perhaps checked daily)
   Rotational Capacity Test: 2 minimum per lot
   Job Inspecting Torque determination: 5 minimum per lot (discard 2 test values)

5. Q. May any fastener components which have been used for any tests (including any Pre-Installation, torque/ tension calibration, RoCap, or determination of Job Inspecting Torque) be reused?
   A. No.

6. Q. Why are torque values from torque-tension tables or formulas not permitted to be used to established proper torque?
   A. Each lot of bolts, nuts, and washers is different (amount and type of lubricant, fit and roughness of threads, and thickness, roughness and type of corrosion-protective coating may vary). A standard table or formula relating torque and tension cannot accurately predict the many variables for a particular lot of fasteners; therefore, values chosen from tables or calculated from a theoretical formula are not acceptable. If an emergency situation arises, contact the fastener specialist at Caltrans Division of Materials Engineering and Testing Services (METS).

7. Q. Who determines the bolt length to be used in a connection?
   A. It is the Contractor's responsibility to provide the correct bolt length, unless the Designer has specified the length in the contract documents. Caltrans specifications require that the final thread stickout shall be a maximum of 1/4" and at least flush with the nut face. This insures full bolt thread engagement with the nut, and also provides a maximum number of threads (at least 3 to 5) within the grip length to insure good ductile capacity of the bolt if loaded in extreme conditions.

8. Q. If a bolt is too long, can additional washers be added?
   A. One washer is required to be placed under the nut (or turned end) of the fastener. Caltrans allows only one additional washer to be added (under the unturned fastener end) as a minor adjustment for proper thread stickout.

9. Q. What should be done when fastener holes in joint plies are misaligned?
   A. The Designer should be contacted and address this condition. It may be permissible to ream misaligned bolt holes up to 1/32" over the diameter normally required for a standard hole. Further reaming to permit use of the next size larger fastener may be acceptable if ample spacing, edge distance, and remaining net section are available in the joint and if allowed by the Engineer. Bolt holes shall only be modified by implementing the placement of holes as stated in Section 55-3.14 of the Caltrans Standard Specifications (Reference 1 of Attachment No. 3).

10. Q. Are warped plates allowed in a bolted joint?
    A. Generally, firm contact between plies cannot be attained during the snugging operation, as required, when warped plates or improper fit-up are present in a bolted connection. Gaps around bolt holes and between plies of a friction-type connection are not acceptable. Proper fit-up of a joint prior to bolting is required. Heat straightening and shimming may be possible corrective measures, which can be used to correct warped plates prior to bolting. The Engineer, however, should use prudent judgement as to the acceptability of any material, given the design considerations. The Paragraphs 3.5.1.14 and 3.5.1.15 of the American Welding Society (AWS) Code D1.5 address the general issue of warped plates for mechanically connected joints and splices.

11. Q. What measures should be taken if Contractor does not handle or store fasteners properly?
    A. Section 8(a) of the RCSC Specification requires that fasteners be stored properly. The Inspector at the job site should immediately notify the Contractor if any fastener components are improperly handled or stored, and should document any instances of improper storage or handling in a diary. Proper handling and storage includes: 1) storing fasteners out of the weather in their original containers, off the ground, preferably in a closed building with a roof, 2) removing only as many fasteners from their original containers as can be installed during a work shift, 3) returning unused fasteners to their original containers in protected storage at the end of the shift, and 4) not altering the original lubricant in any way from the way it was in the as-delivered condition. These requirements are all covered in Section 8(a) of the RCSC Specification.

12.Q. What should be done to fasteners that have become dirty or rusty, or have lost their original lubricant?

A. Fastener components that have not been properly stored may have been exposed to moisture, dirt, or dust, and as a result, may have had lost their original lubricant, or become dirty or rusty. Any changes in the original lubricant or thread condition on most fastener components, especially ones such as Tension Control (TC) fasteners, will affect their torque-tension relationship and how they function and may prevent adequate minimum tension from being attained. Fasteners which have become dirty, rusty or whose original lubricant has changed or been altered should be rejected by the Engineer. Whether the rejected fasteners can be restored to a satisfactory useable condition will vary depending on the degree of degradation and damage. If they are deemed salvageable, how they are to be restored to a useable condition and who can do the restoration will vary, depending on the type of fastener, the type of restoration work required, and the facilities available to the Contractor to rework the fastener components. Each case may require the Engineer to assess what facilities and capabilities the Contractor has available and whether he can do a satisfactory job.

Black fasteners are generally easier to clean and relubricate than zinc-coated ones, and in some cases, this operation can be done by the contractor. Light dust or dirt on fasteners can often be removed and fasteners may be relubricated. Rust on fasteners generally results from improper storage and exposure to moisture. The degree of rust damage and the effect of pitting is often more difficult assess and correct. The degree of rust and pitting will determine whether fasteners are salvageable. Light rust on the male threads can often be removed successfully, and fasteners may be relubricated and reused. Moderate to heavy rust that causes heavy pitting usually cannot be corrected and fasteners should be rejected. Rust on the internal threads of nuts is much more difficult to assess or remove; rusty nuts that cannot be thoroughly cleaned or restored should be rejected. Any restoration of damaged fasteners to their original condition and retesting is the responsibility of the contractor. If the Engineer deems that fasteners can be saved, the Contractor is responsible for assuring that the fasteners are thoroughly cleaned and uniformly relubricated, and then for performing additional pre-installation and rotational capacity tests at his expense, to prove the modified fasteners are acceptable.

Often the Contractor is not equipped to perform satisfactory cleaning and relubrication at the job site. Reworking fasteners that have been rejected due to excessive dirt, rust, or lack of proper lubrication requires certain minimum facilities and equipment. These may include a suitable indoor site, equipment and manpower to 1) thoroughly clean the fasteners (i.e., remove all dirt and rust with appropriate cleaning solvent), 2) apply a uniform amount of suitable lubricant similar to what was originally applied to the fasteners, 3) maintain lot integrity of each fastener component requiring cleaning, and repackage each component and remark containers. The Contractor may wish to rework lots of rejected fasteners, but the Engineer needs to judge whether the Contractor is capable of doing a satisfactory job. If the Engineer does not feel that the Contractor is capable of satisfactorily cleaning and relubricating rejected fastener lots, the Engineer should advise him why.

Each component of a black fastener system is originally provided with a water-soluble oil to protect it from rust and to reduce friction when nuts are being snugged and tightened. For zinc-coated fasteners, only the nuts are lubricated with a special dyed, dry lubricant that is clean to the touch.

The type and quantity of lubricant applied by the original manufacturer to nuts on TC fastener systems is very critical and important. Therefore, any lot of TC fasteners that have been rejected for dirt, rust, or improper lubrication should only be reworked, retested, and recertified by the original manufacturer. Any alteration of the original lubricant by anyone other than the original manufacturer voids any certification or warranty made by the manufacturer of a TC fastener system, and should never be allowed. The Engineer should reject TC fastener systems failing to meet any of the required job site tests. The Contractor may return any rejected lot of TC fasteners to the manufacturer for reworking, retesting, and recertification.

The contractor should be aware that some types of lubricant used on fasteners cannot easily be removed from exposed fastener surfaces after installation and prior to painting the bolts. Some lubricants, such as beeswax, are not water-soluble, are extremely difficult to remove, and may require harsh solvents.

Additionally, lubricants should not be sprayed or applied to bolts that have already been installed in a connection, as the lubricant could seep into the faying surfaces of the joint and result in a loss of friction on faying surfaces of a slip-critical joint.

13. Q. Can a Contractor alter (either add or remove) the original lubricant present on fasteners that he received from the manufacturer?
    A. No. The original lubricant on the fasteners must not be altered. The manufacturer or responsible party for each fastener system has applied a certain amount and type of lubricant to each fastener in a lot, has tested each lot, and certified that the fasteners comply with all appropriate specifications and ASTM requirements. The original fasteners must be properly stored and maintained to preserve their original condition for all preliminary testing, installation, and tension verification checks on each completed joint. The contractor is not permitted to alter any original lubricant on high-strength fastener systems in any way, either for preliminary testing, or before or during installation. If a particular lot of fasteners should fail any of the preliminary tests required and done at the job site, the Engineer should reject the lot.

14. Q. May one type/grade of high-strength fastener be substituted for another?
    A. Generally not. Each grade/type has its own specific material composition, strength and dimensions. Because of smaller head dimensions and shank diameter tolerances, Society of Automotive Engineers (SAE) grades of fasteners (Grades 5 and 8) generally should not be interchanged with ASTM high-strength bolt types. Any request for substitution of a type or grade of bolt different from what was originally specified should be submitted to the Engineer for review prior to acceptance. For further information, contact the high-strength fastener specialist at the Division of METS.

15. Q. If the exterior surface of any steel member is sloped/angled greater than 1:20; can high-strength bolts be used?
    A. Yes; however, if the slope of the exterior face of any member exceeds 1:20 (about 2.9 degrees), relative to the washer-faced bearing surface of the bolt or nut face, a

hardened beveled washer must be used between the exterior face of the sloped steel part and the bolt head and/or nut to compensate for the excessive slope, and reduce the slope(s) to less than 1:20.

16. Q. May high-strength bolts that were used/tightened once, be reused?
    A. Neither ASTM A490 nor galvanized A325 bolts may be reused. Only plain "black" A325 high-strength bolts should be considered for reuse. Reuse of any black A325 bolts and nuts should only be permitted if the Engineer determines the bolts are in good condition, the bolt threads have not been significantly elongated plastically (this can be checked by spinning the nut by hand over the entire length of bolt threads), and each lot of used fasteners is re-tested and passes the pre-installation and rotational capacity tests. All fastener components used for pre-installation or rotational capacity tests, or for determining job inspecting torques shall be discarded.

17. Q. May TC bolts and/or DTI's be reused?
    A. No. Once installed and fully tensioned or used for any type of testing, they must be discarded.

18. Q. Where should a DTI be installed, which way do the bumps face, and how do I determine if the bolt has adequate tension?
    A. The correct preferred position of a DTI is under the bolt head, with the DTI bumps bearing against the underside of the hardened bolt head. Alternate positions are possible, but only when reviewed and approved by the Engineer. DTI bumps must never bear against any soft steel or any turned component. For bolts to have adequate tension, the gaps on zinc-coated DTIs need to be compressed to 0.005" or less (and also need to be greater than 0). The manufacturer's installation procedure should be followed. For more information, obtain appropriate installation instructions from either DTI manufacturer (see Sheet 10 of 10 of Attachment No. 2), or contact the fastener specialist at the Division of Materials Engineering and Testing Services.

19. Q. Who establishes the job inspecting torque and how is it determined?
    A. The Contractor determines the value for inspection torque by testing 5 fasteners, in the presence of the Engineer, in accordance with Section 9(b)(3) of the RCSC Specification. One high and one low reading are discarded, and the remaining three readings are averaged. The Engineer will record the job torque, determine which bolts in the joint shall be inspected, and witness the Contractor performing the actual checking. The procedure shall be performed in accordance with Section 9(b)(4) of the RCSC Specification.

20. Q. Can a contractor partially install ("stuff") some or all fasteners loosely in a joint with the intent of coming back in the near future and completing his tightening operation?
    A. No, absolutely not! The RCSC Specification [Section 8(A)] clearly prohibits this practice. Only as many fasteners as can be completely installed and tensioned during a work shift can be removed from the storage area. This rule helps prevent fasteners from loosing their lubricant and rusting before the tightening operation and tension verification check has been completed. Occasionally an uneducated or unscrupulous contractor will attempt to do this so that he can speed up his operation. Wise inspectors of course prevent this practice and explain why it is a bad thing to do.

21. Q. Why must hot-dip galvanized faying surfaces be hand wire brushed?
    A. Hand wire brushing is required in order to assure that the galvanized surfaces will have sufficient friction between the plates in contact. Using power driven wire brushes can result in polishing of the surfaces, which would reduce the friction between the surfaces and the capacity of the connection.

22. Q. Why is the thread stickout limited to ¼ inch beyond the face of the nut?
    A. If the thread stickout exceeds ¼ inch, the length of full threads within the grip of the joint is very short, and any elongation that occurs in the bolt during tightening is limited to a very small portion of bolt threads within the grip. Excessive thread stickout reduces the ductile capacity of the fastener during extreme unusual combined tensile and shear loading that might take place during an earthquake. In addition, if thread stickout is extremely large, it is possible that the nut would "bottom out" in the transition zone of the threads during tightening and prior to the full tension of the bolt being achieved. In this case, there may be insufficient tension in the bolt although high torque readings may give a false indication otherwise.

23. Q. What level of inspection is required in order to assure that the bolts have been installed properly?
    A. All stages of bolt installation and tensioning must be witnessed in order to assure compliance with the specifications. It is the responsibility of the inspector witnessing high-strength bolting at the job site to thoroughly understand and enforce Sections 2, 3, and 8 of the RCSC Specification. Verifying that the required final torque has been achieved, without witnessing that the snugging and tensioning operations were performed properly, does not guarantee that, after the joint has been completed, each of the fasteners have the minimum tension required.

LIST OF SPECIFICATIONS AND REFERENCES FOR HIGH-STRENGTH BOLTING:

1. Caltrans Standard Specifications, Section 55-3.14, “Bolted Connections”.

2. Standard Special Provisions for high-strength bolting.

3. Project Special Provisions.

4. "**Specification for Structural Joints Using ASTM A325 or A490 Bolts**"**(RCSC Specification)**, Research Council on Structural Connections, American Institute of Steel Construction, Inc., (Allowable Stress Design edition [publication No. S329 (20M596)] or Load and Resistance Factor Design edition [publication No. S345L (30M496)] Chicago, IL, June 3, 1994. Phone No. 1-800-644-2400. ***Available on the OSC Web Site at:***

   http://oscnet.dot.ca.gov/oscnet/

   Note: By reference in the Caltrans Standard Specifications, this RCSC Specification is made a part of all Caltrans construction contracts. The use of high-strength bolts in structural steel connections must conform to requirements in this specification, unless otherwise stated in the contract Standard Specifications or Standard Special Provisions.

5. The following Specifications within the Annual Book of ASTM Standards, Volume 01.08, "Fasteners":
   - ASTM A325 or ASTM A325M, “Structural Bolts”
   - ASTM A563 or ASTM A563M, “Nuts”
   - ASTM F436 or F436M, “Hardened Washers”
   - ASTM F959 or F959M, zinc coated “Direct Tension Indicators”
   - ASTM F1852, “Twist off type TC Bolt Assemblies”

6. The following National Standard titled “Fasteners for Use in Structural Applications”, ASME B18.2.6-1996, published by the American Society of Mechanical Engineers.

7. "High Strength Bolts for Bridges", Report No. FHWA-SA-91-031, May 1991, U.S. Department of Transportation, Federal Highway Administration.

8. Division II- Construction, Article 11.5.6, “Connections Using High-Strength Bolts”, Section 11 “Steel Structures”, of the AASHTO Standard Specifications for Highway Bridges, 16th Edition.

9. AISC Steel Construction Manual.

10. **“Structural Bolting Handbook”**, Steel Structures Technology Center, Inc., 42400 W. Nine Mile Rd., Novi, MI, 48375-4132, (1999 edition) Phone: (248) 344-2910. ***(Contact DSC Headquarters [916-227-8387] to obtain a copy).***

11. “Instruction Manual for Installing High-Strength Bolts with Direct Tension Indicators” (ASTM F959) Inch Series Edition, Turna Sure LLC, 340 E. Maple Ave, Suite 303, Langhorne, PA 19047 (July 1999, 10th edition) Phone: 1-800-525-7193.

Volume II

APPROVED METHODS OF TENSIONING HIGH-STRENGTH BOLTED CONNECTIONS

Introduction

The Caltrans approved methods for tensioning of common high-strength bolt systems consists of two standard methods and two alternative methods. The two standard methods are known as the Turn-of-Nut method and the Calibrated Wrench method. The two alternative methods are known as the Twist Off-Type Tension Control (TC) bolts and the Direct Tension Indicator method. The basic steps for field testing, installation and performing final inspection of the standard methods are very similar to those of the alternative methods.

All fastener systems must pass the required pre-installation test, calibration testing and rotational capacity before being installed in a structure. These tests are performed at the job site by the Contractor and are witnessed by the Engineer. The faying (contact) surfaces of all joint plies must be clean and flat. In many instances, a thin coating of qualified paint or hot-dip galvanized zinc coating may be allowed on faying surfaces. The components to be assembled must fit properly such that the faying surfaces between plies in a joint must have full contact when bolts are installed at a snug condition only. All fasteners in a joint must first be tightened to a snug condition before the final tightening process can begin. In both the snugging and final tightening process, a systematic pattern must be used to tighten each joint, using a crisscross sequence to insure that bolts are evenly tensioned. The final tensioning of A325 fasteners in slip-critical bolted connection must have the following minimum tensions:

Nominal Bolt Diameter	Minimum Tension Values for A325 Fasteners (kips)	
(Inch)	Actual Minimum *	1.05 x Minimum **
1/2	12	13
5/8	19	20
3/4	28	29
7/8	39	41
1	51	54
1 - 1/8	56	59
1 - 1/4	71	75
1 - 3/8	85	89
1 - 1/2	103	108

* Tension values equal to 70 percent of specified minimum tensile strength, rounded to the nearest kip as specified in Table 4, titled "Minimum Fastener Tension for Slip-Critical Connections and Connections Subject to Direct Tension, T_m", of the Research Council on Structural Connections (RCSC) Specification for installing A325 fasteners in slip-critical connections.

** Values are used for calibration and pre-installation testing of all A325 high-strength fastener systems.

Once all fasteners in a joint have been fully tensioned, the joint is inspected. This requires 1) a visual check to insure that plies are in full contact, and thread stickout is in the proper range and is uniform for all fasteners, and 2) the job inspecting torque is applied to 10 % of all fasteners in each joint. Joints should always be inspected immediately after being completed. These same basic procedures are common for all of the approved fastener systems.

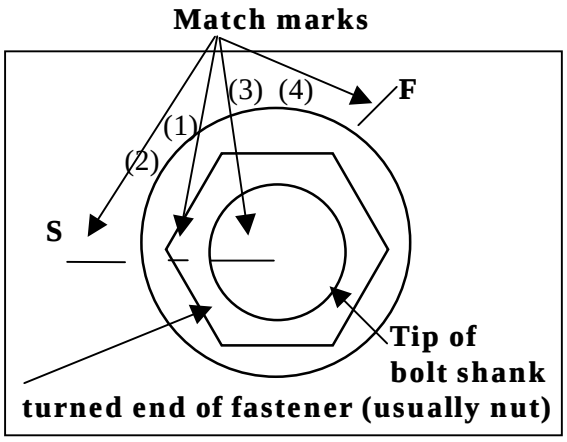
Standard Methods for Installing High-Strength Bolts

The following discussion gives specific information about the two standard methods, Turn-of-Nut and Calibrated Wrench, allowed by Caltrans for installing and checking high-strength bolts:

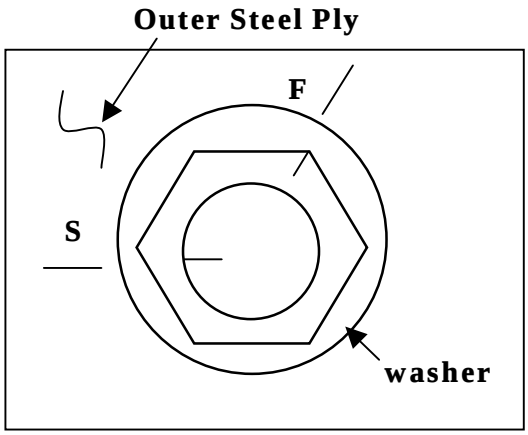
Turn-of-Nut Method

1. First snug tighten all bolts:
When the turn-of-nut method is used, each bolt in a joint must be first brought to a snug-tight condition. At this point, all joint plies should be in firm contact and match marking is done.

2. Match mark all bolts:
When the turn-of-nut tightening method is used to install high-strength bolts, match marking is an important mandatory part of the tightening operation. After snugging, the turned element of all fasteners and the outer plate in the joint are match marked with a felt marker or marking pencil as shown below so that the installer and inspector can see that the nuts have been turned a sufficient amount to adequately tension the fastener. The pictures below show the four initial marks made, and the final position of the marks after tightening has been completed.



Initial position of match marks



Final position of match marks

Note: The two lines on the outer steel ply indicate the start (S) and finish (F) point of the turned element.

In a properly match-marked joint, four marks are made at the turned end of each fastener. These are:

(a). ***A mark on one corner of the nut***. In addition to this mark on one corner of the nut, the outside of the socket used to tighten the nut is usually also marked with a line on its exterior which will be visible during the tightening operation. This mark on the outside of the socket should overlay the hidden mark on the nut corner.
(b). ***A start line, S, put on the outer steel ply*** after all bolts have been snug tightened, and which aligns with the corner mark on the nut.
(c). ***A radial line through the end of the bolt tail***, in line with the start line on the outer steel ply and the nut mark. This radial mark through the bolt tail is important, as it gives a clear indication whether the bolt head turned during tightening (i.e. was properly backed up and kept from rotating during the tightening operation).
(d). ***A finish line, F, on the outer steel ply*** at the appropriate amount of either 1/3, 1/2 or 2/3 of a turn clockwise past the S mark. The location of this (F) mark will vary and depends on the length of the bolt being tightened.

3. Final Tensioning of Fasteners:
The final tightening of the bolt is done as follows:

The socket is positioned so that its exterior mark is aligned with the start (S) mark on the outer steel ply and the mark put on the corner of the nut.The nut is then turned a prescribed amount, depending on the bolt length as shown in Table 5 of the RCSC Specification, until the initial mark made on one corner of the nut lines up with the final mark, F, on the outer ply of the joint. While the nut is being turned, the bolt head (or component of the bolt that will remain stationary) is held with a back-up wrench. The radial mark through the end of the bolt tail should still be aligned with the start mark, S, on the outer ply. If not, this is a clear indication that the bolt head turned during tightening, and the bolt tension may be below the minimum required. This completes the tightening.

The final position of the nut has an allowable tolerance of several degrees with respect to the final mark, F, depending on the size and length of the high-strength bolt. The following are acceptable tolerances:

Bolt Length	Specified Turn	Tolerances Allowed
≤ 4D	1/3 turn (120°)	± 30 °
over 4D but ≤ 8D	1/2 turn (180°)	± 30 °
over 8 D but ≤12D	2/3 turn (240°)	± 45 °

4. Final Check:
Each joint should be inspected as soon as all bolts in the joint have been tensioned. The job inspecting torque check should verify that the bolts in a connection, tightened by the turn-of-nut method, are not below the required minimum tension. Loose bolts may indicate that the bolt heads were allowed to rotate during tightening or the plies of the joint were not in full contact after snug tightening was completed. Therefore, when installing several bolts in a single joint, it is best to snug bolts in at least two tightening stages, and to use a systematic, alternating tightening pattern, starting near the middle of the joint. This process will insure even tension in all bolts when complete.

If the members being joined cannot be brought into firm contact by snugging all bolts, verify that the bolt is the correct length, and that the plies are not misaligned, warped, and do not have burrs and/or irregularities. If plies are misaligned and bolt holes do not line up, the cause of the misfit must be determined and corrected. Further tightening of bolts will generally not correct gaps between plies and around the bolts after they have been snugged, and may result in severely elongated bolts and/or distorted plates.

Calibrated Wrench Method

This tensioning method may be used only when the Contractor's equipment and installation procedures are calibrated daily for each diameter, length, grade, and production lot of bolts.

Torque and/or impact wrenches shall also be recalibrated when significant difference is noted in the surface condition of the bolt or nut threads, or washers.

Calibrated wrenches used for installation shall be set to provide a bolt tension not less than five percent in excess of the minimum tension.

All bolts shall be installed with hardened washers under the turned element, and shall be brought to a snug condition prior to applying the final pretension. Snug tightening shall begin from the middle (or most rigid part) of the connection and progress to the free edges. The tightening operation should be performed such that a systematic (crisscross or alternating) pattern is followed and the same consistent pattern is used for both snugging and final tightening. In some cases, proper tensioning of the bolts may require multiple cycles of systematic partial tightening prior to achieving adequate and even pretension in all the bolts.

When using a torque wrench, the Contractor is required and the Engineer should verify that the torque used on bolts in the structure is consistent with the values determined during the calibration/pre-installation tests done at the beginning of the work shift. In addition, the length of bolt should be checked for compliance to thread stickout limits.

Using a suitable wrench with proper torque capacity for the desired bolt diameter and grade is very important. When the correct size of pneumatic impact wrenches are used, it should take the operator about 10 seconds (after snugging) to achieve the required minimum bolt tension. This condition may result if the time required to tighten a bolt to the minimum required tension is very short (4 seconds or less). It is undesirable to use a wrench which is too powerful for tensioning a particular size or grade of bolt because it can easily result in a fastener whose threaded shank is severely necked down and has been plastically stretched near ultimate capacity; this removes most all of the bolt's residual capacity to stretch and deform plastically without breaking. Using too small, or a worn or broken impact wrench, on the other hand, will not produce the minimum bolt tension required at the recommended 10-second tightening period. Excessive hammering on nuts which results when attempting to tighten a bolt with an inadequate impact tool, (or too little air pressure or air volume) can distort nuts and damage any protective coating, and will still not provide sufficient bolt tension.

The following steps are typical ones used to properly test structural bolts when using a calibrated wrench:

1. The contractor should do calibration/pre-installation testing on a minimum of three bolts, nuts and washers for each diameter, length, grade, and production lot to be used for that day. Testing shall be performed in an appropriate model bolt tension calibrator, according to requirements in the RCSC Specification, Section 8 (d)(2). The contractor must order and use the proper length bolt for a particular joint thickness. Bolts from the same lot that are used in the structure must also be used for verification testing. Additional spacers with the proper center hole diameter must be used to adjust the grip in the bolt tension calibrator, so that two to three threads of stickout is flush with the face of the nut when the nut is finger-tight. Final stickout permitted is between flush and 1/4" past the face of the nut. Appropriate steps, as outlined in the Structural Bolting Handbook should be followed for testing short bolts.

If short bolts are required in the structure and cannot fit into a bolt tension-measuring device, direct tension indicators (DTIs) shall be used to verify adequate tension in the bolts. To determine the appropriate calibrated gap for a particular lot of DTIs, the contractor must furnish longer bolts of the same diameter and grade to be used in the structure, and use them in a bolt tension-measuring device along with DTIs. Once an appropriate calibrated DTI gap is established, the same lot of DTIs shall be used to determine torque or impact wrench setting for the short bolts installed in steel plate shimmed to the appropriate thickness to simulate the actual joint. The short high-strength bolts shall then be tensioned in a simulated joint to produce the same calibrated gap verified with DTIs from the same lot (Reference "Direct Tension Indicator Method) and a torque value read at that gap. The average of the three torque values shall be the installation torque for that lot of short bolts and for that day.

2. First, the bolt must be brought to a snug condition. For the initial snugging, a spud wrench, impact wrench, or bar and socket may be used. The same tools used when installing high-strength bolts in the actual structure shall be used during installation testing.

3. Final tightening should follow one of the two following procedures:

a. Procedure to be used with impact wrench:

(1). Tighten the bolt by turning the nut until the wrench "cuts out". Verify that the tension achieved, as read on the bolt tension-measuring device, is at least 1.05 times the required bolt tension.

(2). Check the degree of turns on the nut to make sure it does not exceed the corresponding tolerance for the "turn-of-nut" rotation. If the amount the nut has been turned has exceeded the maximum rotation allowed, discard the assembly. A new assembly should be tested with the impact wrench torque value adjusted to correspond to the required bolt tension.

(3). The high-strength bolt assembly shall be tested to ensure that the minimum tension is attainable by the installation crew and the tools being used, without exceeding the prescribed rotation.

b. Procedure to be used for torque wrench:

(1). Tighten the bolt by turning the nut until the tension on the bolt is at least 1.05 imes the required bolt tension.

(2). Reading the dial on the torque wrench, measure the moving torque while turning the nut an additional 5 degrees in the tightening (clockwise) direction. This is the torque value that should be recorded.

(3). The average of the three values or the highest acceptable value should be used as the installation torque for this day.

Alternative Methods for Installing High-Strength Bolts

The following discussion gives specific information about the two alternative methods, Twist Off-Type Tension Control (TC) Fastener Assembly and Direct Tension Indicator, allowed by Caltrans for installing and checking high-strength bolts:

Twist Off-Type Tension Control (TC) Fastener Assemblies

All twist-off type tension control (TC) fastener assembly consists of a unique bolt having a splined end, a nut and a hardened washer. The head on the bolt is commonly domed or rounded, but may be manufactured with a hex shape. TC fastener assemblies are produced and shipped by the manufacturers as a precisely engineered and fully tested system. They must comply with requirements in the ASTM F1852 specification. Lubricant types and amounts and machining tolerances may be different from one lot to another, and consequently, the component parts may not be interchanged or altered in any way. Each assembly lot must be used only in the as-delivered, factory-lubricated condition. TC fasteners are installed using an electric wrench having a specially designed planetary chuck. This planetary chuck has dual sockets that engage both the nut and splined tail of the bolt at the same time and turn one relative to the other chuck until the splined tail on the end of the bolt breaks off.

When inspecting a TC fastener installation to ensure a that a quality job is being done, a number of things must be checked: the initial job-site testing of the fasteners must be carefully observed and checked, the installation procedure required by the manufacturer must be reviewed, proper storage of the fastener assemblies out of the elements must be constantly checked, the tensioning operation must be carefully monitored while in progress, and the final tension of at least 10% of the fastener assemblies must be checked using a job inspecting torque. Just verifying at the end of the job that the splined end of each bolt has sheared off is not adequate. This only signifies that at some time during installation, the assembly was subjected to a torque adequate to cause the shearing of the splined tail, not that the final tension in each fastener is adequate.

As with other fastener assemblies, representative samples of TC fastener must be taken from each lot and pre-installation tests run at the beginning of the job. Successful completion of these pre-installation tests will to assure that 1) the installer knows the proper procedure to install the fasteners and follows the manufacturers instructions, 2) the actual equipment he is using to install the fasteners works properly, and 3) the fasteners provide the minimum tension as specified in Section 8 (d)(3) of the RCSC Specification.

When observing pre-installation tests, the following should be verified:

1. A representative sample of not less than three bolts of each diameter, length, grade, and lot shall be installed and tensioned by the Contractor at the job site in a bolt tension calibrator. The Contractor's installer shall demonstrate that each assembly develops a tension not less than five percent greater than the tension required by Table 4 of the RCSC Specification.

2. When testing a TC bolt having a domed head in a bolt tension meter, a flat bushing specifically made for testing the domed tension control bolts must be used under the domed head. These special bushings are not normally furnished as standard parts with bolt tension calibrators. A different size of bushing is required for each bolt diameter being tested and can be purchased through the manufacturer (such as Skidmore-Wilhelm) of the bolt tension calibrator.

3. The TC fastener assembly shall be tested using one flat hardened washer (furnished by the manufacturer of the TC fastener assembly), under the nut (turned element).

Each TC fastener assembly shall first be snugged using the same effort and snugging equipment that will be used on the final structure. During the snugging operation, if the spline breaks off, the bolt shall be removed and the bolt tension at snug tight checked. If the tension at snug tight exceeds 50% of the minimum required tension load, the effort used to snug tighten the fastener should be reduced and new pre-installation tests run.

4. If when running the pre-installation tests, the TC bolts are too short to fit into a bolt tension-measuring device, direct tension indicators (DTIs) must be used to verify the proper tension. First a calibrated DTI gap needs to be determined using three bolts long enough to fit into a Skidmore, tightening each until a load of 1.05 times the minimum preload value has been attained, and then, using tapered feeler gages, determining an average gap value for the compressed DTIs. Once an average calibrated gap value has been determined for three DTIs, the same lot of DTIs shall be used in conjunction with short TC bolts in a simulated joint having the same grip as in the actual structure. When short TC bolts have been installed (tail has been snapped), the DTI gap must be equal or less than the calibrated value determined by using long bolts in a Skidmore bolt tension calibrator. This confirms that the fastener tension is equal to or greater than the minimum required.

Rotational capacity testing is also presently required by Caltrans for this system and for this testing, conventional installation tools should be used (to prevent the splined end from being sheared off).

When tension control fastener assemblies are installed in a structure, the following procedure must be followed:

1. TC fastener systems must always be properly stored out of the weather and maintained in the original condition as supplied by the manufacturer, or else the fastener tension will change and problems will arise.

2. When assembling a TC-bolted connection as with other fastening systems, a TC fastener assembly must be installed in each of the holes of the connection.

3. The bolts shall be systematically snugged (preferably using a conventional tightening tool commonly used for a snugging operation – not an electric TC fastener installation tool) to bring all plies of the joint into firm contact and without yielding or fracturing the splined tails of the fasteners. If the TC fasteners are incorrectly installed and full tensioned in a single continuous operation, they will give a misleading indication to the inspector that all the fasteners are properly tightened. However some of the initially tensioned fasteners may not be. If the plies of the joint are not in firm contact after snugging bolts, then the cause needs to be determined and corrected.

4. Finally during the final tightening (tail snapping operation), each assembly is tightened following a systematic, crisscross pattern starting from the center of each joint.

After installation has been completed and there is any question about whether there is adequate tension in the TC fasteners, the following should be done:

- Uniform and proper thread stickout should be checked. After the spline has broken off, a partially threaded section (approximately 1/8") typically remains; these partial threads at the broken end of the TC bolt are not to be considered as part of the thread stickout. Therefore after installation, the actual length of the projecting bolt stub should extend at least 1/8" beyond the outer face of the nut to a maximum of 3/8".

- The contractor should determine a job inspecting torque value.

- A minimum of 10% of the TC fasteners must be checked using a torque wrench for adequate minimum preload.

Direct Tension Indicators (DTIs)

A direct tension indicator (DTI) is a special device used in conjunction with each high-strength bolt to insure proper tension in the bolt has been attained. DTIs have a number of evenly spaced bumps protruding on one side that are compressed against a hardened surface in a controlled manner. As the bolt is tightened, the bumps are crushed. When they reach a prescribed crushed height (0.005" for bridge and sign structures), the high-strength bolt has been sufficiently tensioned.

Basic steps for field testing of DTIs in a bolt tension calibrator (e.g. Skidmore) are as follows:

1. Test three DTIs of each diameter, grade and production lot, plus three sample bolts, nuts and washers. It is not a requirement that this test be conducted on each separate lot of bolts and nuts. Each DTI, along with sample fasteners, is called a "test assembly".

2. Testing DTIs in a bolt tension calibrator requires the use a special flat bushing and flat hardened washer. The bushing available from the bolt tension calibrator manufacturer (Skidmore-Wilhelm) must be used under the nut (or turned element). DTIs are normally placed under the bolt head, with the bumps bearing directly against the underside of the bolt head (non-turned element).

3. Add spacers and washers under the nut, as necessary, to adjust thread stickout from zero to two threads beyond the face of nut, when the nut is finger-tight.

4. Testing or installing DTIs in a bolt tension calibrator is a two-person operation. While tightening the nut, the bolt head must be prevented from turning.

5. First snug the bolt with a DTI as will be done in the actual structure. In the snug condition, no gap on a DTI may be less than 0.015". Use a 0.015" feeler gage to check for gaps less than 0.015" at snug.

6. Then tighten the nut until the bolt tension as read on the bolt tension calibrator is equal to 1.05 times the minimum required bolt tension. Check how many gaps around the perimeter of the DTI the tapered feeler gage enters. It should enter 1/2 or more of the total number of gaps around the DTI.

7. Continue tightening the fastener until the number of gaps which a 0.005" feeler gage won't enter equals or is greater than that shown in Column 4 of the table on Sheet 10 of 11 in Attachment 2 of BCM 170-2.0. The tension in the bolt as measured by the calibrator must be less than the minimum tensile strength of the bolt.

On the actual structure, verify that bolt heads are held stationary with a back-up wrench when nuts are being turned. In addition, check that all of the bolts in the connection are systematically snugged starting from the center of each joint, and the faying surfaces of all joint plies are in firm contact prior to performing the final tensioning of the bolts.

When installing a DTI, the protrusions shall always be positioned so that they bear against a hardened surface (normally the underside of the bolt head) that must be held stationary as the bolt is being tightened. Before bolts are permitted to be installed in the structure, a representative sample of at least three assemblies, of each diameter, grade, and lot shall be tested in a calibrated bolt tension-measuring device. The test assembly shall include a flat, hardened washer under the turned element. By doing the pre- installation test (also called field test in ASTM F959) the installation crew shall demonstrate that, using the same bolts, snugging and installation tools, and techniques to be used on the actual structure, and compressing the DTI protrusions to an average gap of 0.005", it will achieve a tension no less than 1.05 times the specified minimum bolt tension. This requirement is in Section 8 (d)(4) of the RCSC Specification.

When high-strength bolts are installed in the structure in conjunction with DTIs, the fasteners shall be installed in all holes of the connection and tightened starting from the center (most rigid part) of a joint in a systematic pattern, until all plies of the joint are in firm contact.

When an actual joint is being assembled, the fasteners should be checked to ensure they are uniformly snug. A snug tight condition is indicated by partial compression of the DTI bumps. After snugging bolts, any DTI which has been compressed such that any gap less than 0.015" shall be removed and replaced with a new indicator.

Once all fasteners in a joint have been snugged, the fasteners shall then be systematically tensioned, as was done during snugging. In some cases, proper tensioning of the bolts may require multiple cycles of systematic partial tightening prior to achieving even final bolt tension in order to bring bumps in all DTIs to a uniform gap. When inspected after installation, the minimum number of gaps refusing a 0.005" tapered feeler gage shall be as follows: If all gaps have been reduced to 0 after installation has been completed, the DTI shall be removed and a new DTI and fastener installed.

DTIs should not be used when over-sized holes are present, unless approved by the Engineer and manufacturer of the DTI. If approved, special flat hardened washers must be used.

If a DTI cannot be placed under the bolt head (stationary element) due to unusual field conditions, contact the high-strength fastener specialist at the Caltrans Division of Materials Engineering and Testing Services (METS) for assistance. For DTIs approved for installation under the turned element, special hardened washers with a small inside diameter may be necessary, and can be obtained from the DTI manufacturer.

Attachment No. 1 contains answers to frequently asked questions regarding associated tools and equipment used in high-strength bolting.

COMMON QUESTIONS AND ANSWERS CONCERNING TOOLS USED IN HIGH-STRENGTH BOLTING

Bolt Tension Calibrators:

1. Q. What type of equipment should be used to perform torque and tension checks on high-strength fasteners?

   A. Skidmore-Wilhelm (models MS, M, or ML) or Norbar bolt tension calibrators are assigned to some ACM's to do quality assurance testing.

Typical bolt tension calibrator and steel case

The appropriate model Skidmore should be used, depending on the shortest length bolts to be tested (see chart below).

Minimum Bolt Length (inches) which can be tested in Various Models of Skidmore-Wilhelm Bolt Tension Calibrators

Nominal Bolt Size,	Model of Bolt Tension Calibrator				
	M	ML	MS	H	K (Bench Model)
1/2 – 13	2.25	2.250	2.000		
5/8 – 11	2.50	2.500	2.000	2.750	
3/4 – 10	2.75	2.750	2.000	3.000	
7/8 – 9	3.00	2.750	2.250	3.000	
1 – 8		3.000	2.500	3.250	
1 1/8 – 7		4.750	3.250	5.250	
1 1/4 – 7		5.000	3.375	5.500	5.500
1 3/8 – 6				5.500	5.500
1 1/2 – 6				5.750	5.750
Max. Tension Capacity	80K max.	110K max.	90K max.	170K max.	225K max.
Weight	65 lbs. +				180 lbs.

There are also some older Norbar bolt tension calibrators available; check to make sure equipment has been calibrated within the past year, and is within the required accuracy limits.

2. Q. Are all of the necessary parts available with the basic bolt tension calibrators?
   A. Probably not. To test domed head TC bolts and all DTIs, a special set of flat bushings is required which is not normally furnished with the standard calibrator equipment. Flat bushings are readily available from the Skidmore-Wilhelm Mfg. Co. and are shown below.

Special Flat Bushings Required to Test TC Bolts and DTIs

Nominal Bolt Size, inch	**Model of Bolt Tension Calibrator**					
	M and ML Bushing #/ Approximate Cost		MS Bushing # / Approximate Cost		H Bushing # / Approximate Cost	
1/2	MT-608	$15.00 ea.	MS-608	$30.00 ea.	HT-708	$20.00 ea.
5/8	MT-610	$15.00 ea.	MS-610	$30.00 ea.	HT-710	$20.00 ea.
3/4	MT-612	$15.00 ea.	MS-612	$30.00 ea.	HT-712	$20.00 ea.
7/8	MT-614	$15.00 ea.	MS-614	$30.00 ea.	HT-714	$20.00 ea.
1	MT-616	$15.00 ea.	MS-616	$30.00 ea.	HT-716	$20.00 ea.
1 1/8	MT-618	$20.00 ea.	MS-618	$35.00 ea.	HT-718	$20.00 ea.
1 1/4	MT-620	$20.00 ea.	MS-620	$35.00 ea.	HT-720	$20.00 ea.
1 3/8					HT-722	$20.00 ea.
1 1/2					HT-724	$20.00 ea.

Note: The model MS Skidmore bolt calibrator was developed to test bolts which are shorter than can be tested in the Models M or ML calibrators. In Skidmore-Wilhelm's development/design process, the front plates of the Model MS were made thinner. Skidmore-Wilhelm has pointed out that when testing TC bolts, the thinner MS plates will dish slightly when tensioning the fastener. This dishing may reduce the contact area between the nut face and washer, and may cause a slight increase in the torque required to rotate the nut. This may cause a reduction in tensile load read on the bolt calibrator dial at which the spline shears off. Skidmore recommends ordering and using the thicker plates for the MS model if the tension control fasteners are not meeting minimum tension requirements by 5 to 10%. The Skidmore-Wilhelm Mfg. Co. should be contacted immediately (216-481-4774) if use of their Model MS calibrator results in too low tension values when the spline shears off.

In addition to bushings, spacers of the proper length and inside diameter may also be required when testing long bolts to adjust thread stickout to between 0" (flush) and 1/4".

3. Q. Should Caltrans provide the bolt tension calibrator for use on construction projects?
   A. No. The Contractor is to perform all pre-installation testing, rotational capacity testing, and inspection of completed joints at the job site, and shall provide all necessary tools and appropriate calibrated equipment, including torque wrenches (dial or digital readout only), bolt tension calibrators, impact wrenches, sockets, and torque multipliers to do so. The Engineer is to witness preliminary calibration, testing, select

fasteners to be inspected in a completed joint and witness joint inspection. Caltrans has Skidmore-Wilhelm bolt tension calibrators available for use by Caltrans employees doing quality assurance inspection.

4. Q. How often should the bolt tension calibrator be recalibrated and adjusted and who is qualified to do this?
   A. Recalibration and adjustments should be performed at a minimum annually, by a certified testing facility using equipment that is traceable to the National Institute of Standards and Technology at the Contractor's expense. Required accuracy, after calibration is within ± 2% of the actual load. The following is a list of acceptable laboratories that are qualified to recalibrate and repair bolt tension calibrators:

Manufacturers* and Laboratory Test Facilities for Bolt Tension Calibrators			
1)	Almay Labs 1415 Newton St. Los Angeles, CA 90021 Phone: (213) 746-1555	4)	Advanced Witness Series, Inc.** 910 Bern Ct. #100 San Jose, CA 95112-1237 Phone: (408) 453-5070
2)	Technical Services Group P.O. Box 250 Alameda, CA 94501 Phone: (510) 522-8326	5)	Mountz, Inc. 1080 North 11th Street San Jose, CA 95112 Phone: (408) 292-2214
3)	Skidmore-Wilhelm * Manufacturing Co. 442 South Green Rd. Cleveland, OH 44121 Phone: (216) 481-4774	6)	Norbar Torque Tools Ltd. * Beaumont Road, Banbury Oxon, OX167XJ United Kingdom Phone: 44(0) 1295 270333

** Local representative and calibration/repair center for Norbar equipment

5. Q. Where can a field Engineer get more information about bolt tension calibrators?
   A. Information and literature can be attained directly from the manufacturers or their representatives (see the listing in Question 4 above).

Torque Wrenches:

1. Q. May the Contractor perform testing or installation of high-strength bolts with a "click-type" torque wrench?
   A. No. Either a dial or digital gauge torque wrench is required to accurately determine installation torque, perform RoCap tests, and determine job inspecting torque for inspection of a completed joint. When performing a RoCap test or determining proper job inspecting torque at the jobsite, the contractor cannot read the particular torque value at a given bolt tension when a click-type torque wrench is used. Also, generally the accuracy of a click-type torque wrench is not adequate to meet Caltrans accuracy requirements.

Dial-type torque wrench

Digital-type torque wrench

Acceptable types of torque wrenches

2. Q. What is the "best practice" when using torque wrenches?
   A. A torque wrench is properly used when the following concepts are followed:

   - Proper installation procedures must be verified and torque values reestablished at least once each working day for each bolt diameter, length, and lot. A hardened washer must be used under the nut (turned end).

   - Always use a proper size torque wrench with adequate torque capacity or use a smaller wrench in conjunction with a torque multiplier, when necessary. Never use a torque wrench that is too small for the job. If the torque limit of a wrench is exceeded, the wrench is generally ruined and cannot be repaired.

   - Operating a large torque wrench is generally a two-person operation - one person to pull on the handle and one to read the dial. The person pulling should use a smooth motion. An extender handle of adequate length should be utilized to reduce effort required to turn nuts and to insure a smooth turning motion (A "jerky" motion generally results when the lever arm is too short). Difficulty attaining adequate tensile loads on large high-strength bolts (i.e., 1-1/8" to 1-1/2") generally indicates the need to use a torque multiplier in conjunction with a torque wrench.

   - It is generally not good practice to use a torque wrench to undo a bolt. A torque wrench is a delicate instrument, and the mechanism of some is not designed to be loaded in the reverse (counter-clockwise) direction. It is much better to use an impact wrench or breaker bar for loosening or unloading tensioned fasteners.

   - When performing RoCap tests, never use a torque wrench to perform the final portion of the test that requires that the bolt be rotated a specified large number of turns. This portion of the test may require a torque level that will severely overload a torque wrench normally used for routine fastener tensioning and cause irreparable damage. A breaker bar or pneumatic wrench in combination with a torque multiplier should be used for this last phase of the RoCap test.

3. Q. Where can I obtain information about or purchase a satisfactory torque wrench?
   A. From one of the companies from the following chart:

Torque Wrench Manufacturers* and Calibrator Companies			
1)	Proto* 14117 Industrial Park Blvd., N.E. Covington, GA 30209 Phone: (770) 787-3800	4)	Armstrong Tools* 5200 W. Armstrong Ave. Chicago, Illinois 60646 Phone: (312) 763-3333
2)	Snap-On* 6632 Fig St., Unit B Arvada, CO 80004 Phone: (888) 762-7972	5)	Advanced Witness Series, Inc. 910 Bern Ct. #100 San Jose, CA 95112-1237 Phone: (408) 453-5070
3)	Mitutoyo* 16925 Gale Ave. City of Industry, CA 91745 Phone: (818) 961-9661	6)	Mountz, Inc. 1080 North 11th Street San Jose, CA 95112 Phone: (408) 292-2214

4. Q. How often should a torque wrench be recalibrated and adjusted?
   A. At least once a year. It must be recalibrated and adjusted more often if dropped/mishandled.

5. Q. Who is qualified to perform the recalibration and adjustments, and what information/certificates are required?
   A. A certified testing facility must have calibration equipment traceable to National Institute of Standardized Testing (NIST) standards and must perform calibration checks and any adjustments on equipment. If equipment cannot be adjusted so that its accuracy is within specifications, it is not permitted to be used.

6. Q. What accuracy is required for a torque wrench to be considered acceptable?
   A. Accuracy of torque wrenches shall be within 2 percent of the actual torque value, with a minimum of 4 verification readings evenly spaced over a range of 20 to 100% of full scale. If there are any questions about the accuracy of the contractor's torque wrench, a copy of the latest calibration check after any adjustments were made should be required.

7. Q. What do the terms *break torque* and *moving torque* mean?
   A. Break Torque: The torque value required to initially start a nut in motion from a stationary position.
   Moving Torque: Torque measured while the nut is in motion.

8. Q. Does Caltrans provide torque wrenches?
   A. Yes, but only for Caltrans quality assurance testing or checking Contractor's values, not for work that is required to be done by the Contractor.

9. Q. When using the calibrated wrench installation method, why is it necessary to determine an installation torque value for each lot of fasteners every day?
   A. Thread conditions (roughness, coating type and thickness, and pitch diameter), the type and amount of lubricant, and storage and weather conditions may be different for each lot of bolts. These variables can have a huge effect on the relationship between torque and tension, especially when using the calibrated wrench method to install high-strength fasteners. Therefore the RCSC Specification requires that certain installation procedures must be performed and appropriate installation torque values

must be determined on a daily basis, using fastener samples from each lot of bolts to be installed in the structure.

Calibrated Wrenches:

1. Q. How often must a calibrated wrench be adjusted and checked to produce proper bolt tension?
   A. At a minimum, the tension value produced by a calibrated wrench must be checked daily at each job shift for each bolt diameter and length to insure the cutoff setting is correct. If a different lot of fasteners is used, or operators, length of air lines, tool being operated, or thread conditions on the fasteners change, the calibration of the wrench must be checked and perhaps recalibrated.

2. Q. Which types of calibrated wrenches are acceptable?
   A. Besides a dial or digital torque wrench, a pneumatic, hydraulic, or electric wrench with an adjustable control unit which can be set to positively shut off at the desired torque is acceptable. A standard impact wrench without a positive cutoff is not acceptable.

Acceptable types of calibrated wrenches

3. Q. What are some equipment variables that effect the final product?
   A. Compressor size and condition, the length, number, and size of air lines, air volume demand of other equipment being operated at same time, condition and size of pneumatic impact wrench, and adjustment settings all can affect torque output of an impact wrench.

4. Q. Why does an impact wrench need to be adjusted and checked so frequently?
   A. Experience has indicated that operator "feel", condition and size of compressors, air lines, air tools, air pressure variations and the number of tools run simultaneously on a single manifold are all variables that account for differences of bolt tension at the snug tight condition, as well as the final tension in the high-strength bolts. The influence of these numerous factors must be checked to insure consistent and accurate bolt tension.

5. Q. Why should it take approximately 10 seconds to fully tension A325 bolts?
   A. A wide variety of brands of calibrated impact wrenches with varying torque capacities are available and appropriate for various sizes of high-strength bolts. When using the correct size of impact wrench with A325 bolts, it takes about 10 seconds (after snugging) to achieve the minimum required bolt tension. Using *too large* of a calibrated wrench for a given size of bolt can result in the threaded shank of the bolt being elongated with the applied stress far exceeding the elastic limit (well into the plastic zone), resulting in very little remaining ductility. Using an impact wrench which is *too small* or *worn* will result in low fastener tension, wasted time and noise

during installation, and excessive hammering on nut, causing damage to any protective coating.

Torque Multipliers:

1. Q. What is this tool and when is it used?
   A. The torque multiplier is a tool that amplifies a small input torque by gear reduction to produce a large output torque. It is commonly used in conjunction with a torque wrench, to reduce the tightening effort needed for testing, installing, and inspecting larger sizes of high-strength bolts. By using a multiplier, less tightening effort can be applied using smaller input wrenches and tools, and smoother and safer tightening operation results. This prevents accidents caused by dangerous but commonly used installation practices of overexertion on a short handled manual torque wrench, or hanging on the end of a long, heavy cheater handle. Also by using a multiplier, permanent damage to smaller torque wrenches which can easily be stressed above their minimum torque rating, can be avoided. An anti wind-up ratchet is a desirable optional feature available on many of the better multipliers.

Acceptable types of torque multipliers

2. Q. Are different sizes available?
   A. Yes. Multipliers are available with various multiplication factors and input and output drive sizes. (E.g., 1:4, 1:5, 1:10, 1:15, 1:25, 1:75, 1:100, etc.)

3. Q. Who manufactures and sells multipliers?
   A. Check the catalogues from following companies: Mountz, Norbar, Proto, Snap-On, and Advanced Witness Series, Inc. Phone numbers for these companies are included in the charts shown in Question 4 on Sheet 3 of 11, Attachment No. 2 of BCM 170-2.0, and in Question 3 on Sheet 5 of 11, Attachment No. 2 of BCM 170-2.0. For further information contact Dan Thomas in the Division of Structure Construction Headquarters or the fastener specialist at the Division of Materials Engineering and Testing Services.

4. Q. How often does a multiplier need to be recalibrated?
   A. At least once a year. When a torque wrench is used in conjunction with a torque multiplier, the two should always be used together with a bolt tension calibrator to determine an accurate input torque/bolt tension relationship for installing and inspecting fasteners.

Electric Installation Tools with combo Spline/Nut Socket for Installing (TC) Bolts:

1. Q. Do all manufacturers of electric installation tools offer the same size of tool?
   A. No, not all electric tools (called Shear Wrench Tools) for installing TC fasteners have the same capacity. Normal electric installation tools require from 11 to 15 inches of working space. Some tools are designed with a right-angled drive for working in tight quarters and have a working clearance of about 7-1/2 inches. In addition, different manufacturers of electric installation tools and sockets for TC fasteners may produce

spline sockets with slightly different internal dimensions. Variations in spline dimensions of TC bolts made by different fastener manufacturers and the actual thickness of zinc coating on the spline section may vary slightly. These conditions can prevent a tool socket from fitting onto the fastener spline. A tight socket clearance or heavy zinc coating on the spline may also prevent easy ejection of the broken spline from the inner socket.

Electric tool for installing TC bolts

2. Q. What variables can effect the satisfactory installation of a TC bolt assembly?
   A. This system is dependent on close manufacturing tolerances (bolt and nut thread dimensions, groove dimensions at tip), steel chemistry, amount and type of nut lubricant, and consistent heat treating all can affect the final bolt tension. Typical steps required with all other high-strength bolt operations, including proper joint fit, snug tightening, tensioning all fasteners evenly in an alternating pattern, are also required when installing TC bolts.

3. Q. Do you need to check the final TC bolt tension with a calibrated torque wrench when fastener installation has been completed?
   A. Yes. Like any normal high-strength bolted connection, joints can be improperly fit and TC bolts can be improperly installed. Plates in joints must flat, should have full bearing after snugging, and all fasteners in a joint must first be evenly snug tightened, and then fully tensioned in stages and in a patterned sequence, or else like any other fastener, some of the TC bolts initially tightened will be loose after the joint has been completed. As required with other tensioning methods, a job inspecting torque must be established and 10% of bolts in each connection (or a minimum of 2 bolts) should be checked for adequate tension immediately after tightening of a joint has been completed.

4. Q. How does one know if TC bolts have been tightened to at least the minimum required tension?
   A. By performing pre-installation tests on each lot, diameter, and length of fasteners, observing that proper installation procedures are followed, and inspecting 10% (or a minimum of 2) bolts in each joint using the appropriate job inspecting torque value.

5. Q. Should all TC bolts in a joint be taken to a snug condition?
   A. Yes. First, all TC fasteners should be first snugged in a connection <u>without snapping off the splined tails</u> before any final tightening is done. Use of a standard impact wrench, torque wrench, or spud wrench is recommended for the snugging operation. If the snugging operation is skipped, and instead, all TC fasteners are fully tensioned without first drawing all plies together, fasteners initially tightened may be loose. If the splined end of a TC bolt has been sheared off, it merely signifies that at some time during the tightening operation, the fastener has been subjected to sufficient torque to cause shearing of the spline. It does not necessarily mean that the final tension is adequate. An uninformed inspector who looks at a completed job may not be aware that just because the tails of all TC fasteners are broken, it does not necessarily mean that all TC fasteners have adequate tension.

<u>Tapered Feeler Gages Required for Measuring Gaps between Bumps on Direct Tension Indicators (DTIs):</u>

1. Q. Why do feeler gages need to be tapered in order to inspect for proper gap in an installed DTI?
   A. Feeler gages used for inspecting DTIs must be tapered and have narrow tips so that they can fit into each gap between closely spaced DTI bumps.

Gage from TurnaSure LLC

Gage from Applied Bolting Technology

<u>Typical tapered feeler gages for inspecting an installed DTI</u>

2. Q. Which thicknesses of tapered feeler gages are used for inspecting DTIs installed on bridge or sign structures: 0.015" or 0.005"?
   A. Two thicknesses of these special tapered feeler gages, 0.005" and 0.015", are commonly used for inspection. The Contractor should have them available and use them during pre-installation testing, installation, testing of short bolts, and determination of job inspecting torques. Caltrans inspectors should also have them handy. The 0.015" feeler gage can be used to inspect DTI gaps after snugging. The 0.005" tapered feeler gage is used to verify adequate DTI crushing during final tensioning for bridge and sign structures. After installation of a DTI is complete, the 0.005" feeler gage tip must be refused in at least 1/2 of the gaps, but all the bumps must not be fully compressed to a "0" gap. A whole set of 26 tapered feeler gages is also necessary when determining a "calibrated gap" for performing pre-installation tests and a job inspecting torque for short bolts (see pp 17 & 18 of the Structural Bolting Handbook).

3. Q. Where can I obtain tapered feeler gages?
   A. The 0.005" and 0.015" tapered feeler gages are available from either DTI manufacturer (see Question 5). Sets of tapered feeler gages (26 leaves) are available from Starrett (model 66T) [phone: 617-249-3551]; Mitutoyo (model 950-242); or McMaster-Carr [phone: 310-692-5911]. The widths of the tapered leaves may have to be trimmed down to match the taper and tip width of the DTI manufacture's gages, so that they will fit between the DTI bumps.

4. Q. How many gaps must refuse the tapered feeler gage for the DTI to be acceptable?
   A. The following chart shows the number of gap refusals required for each size of A325 bolt to insure that the minimum tensile strength has been attained:

Bolt Diameter, inch	Total Number of Gaps on DTI	1.05 x Minimum Bolt Tension, kips	Minimum Gap Refusals For Minimum Bolt Tensile Strength	Minimum Bolt Tensile Strength, kips
1/2	4	13	2	17
5/8	4	20	2	27
3/4	5	29	3	40
7/8	5	41	3	55
1	6	54	3	73
1 1/8	6	59	3	80
1 1/4	7	75	4	102
1 3/8	7	89	4	121
1 1/2	8	108	4	148

5. Q. What companies manufacture DTIs?
   A. In the United States, only the two following companies currently make DTIs:
   1. Applied Bolting Technology Products, Inc.
      P.O. Box 255
      Ludlow, Vermont 05149-0255

      Telephone:..... 802-228-7390
      800-552-1999
      Facsimile:..... 802-228-7204

   2. TurnaSure LLC
      340 E. Maple Avenue
      Suite 303,
      Langhorne, PA 19047

      Telephone:..... 215-750-1300
      800-525-7193
      Facsimile:..... 215-750-6300

Note: Both DTI manufacturers furnish free installation instructions and tapered feeler gages.

BRIDGE CONSTRUCTION MEMO 170-4.0

STRUCTURAL STEEL

July 1, 2001

Volume II

STRUCTURAL STEEL WORKING DRAWINGS

Introduction

The procedure for review and approval of structural steel working drawings is a coordinated effort between Design and Construction personnel. The designer, with input from construction, has the primary responsibility for approval of the working drawings. In the case of projects designed by a Local Agency or Consultant the designer of record, in conjunction with Structure Design and the Design Oversight Engineer, has the primary responsibility for approval of the working drawings. However, the Shop Plan Clerk, in the Structure Design Documents Unit, will furnish the Local Agency Designer or Consultant with two copies of the working drawings.

Working Drawings

Attached is Memo to Designers 12-1 (Attachment No. 1) "Review of Working Drawings - Steel Structures". The Memo to Designers, which is a cooperative effort of Design and Construction, covers the procedures required for review and approval of working drawings including responsibilities of the Structure Representative on a construction project. The procedure for submittal of plans and working drawings is for the Contractor (subcontractor or fabricator) to submit all documents directly to the Office of Structure Design, Documents Unit, Mail Station 9, 1801 30th Street, Sacramento 95816. The original submittals and any resubmittals shall be submitted to the Documents Unit. The Structure Representative should ensure that the contractor submits all documents to the Documents Unit in a timely manner.

The Documents Unit is responsible for administering the working drawing review and approval procedure during all phases of the approval procedure. The Documents Unit maintains a record of all working drawings submitted and distributes copies to the required individuals. This relieves the Structure Representative of the tedious administrative details required in distributing and coordinating the review and approval process.

The responsibility for checking working drawings is shared by the designer of record and the Structure Representative. The Structure Representative shall make all effort to coordinate directly with the designer. The working drawings shall not be returned to the Contractor until the designer of record has discussed and resolved all comments with the Structure Representative. The comments that are returned to the Contractor must be acceptable to both the Designer and the Structure Representative.

The Structure Representative shall ensure that the final working drawings are submitted by the Contractor to the Documents Units, in accordance with Section 55 of the Standard Specifications, prior to the acceptance of the contract. The Structure Representative can verify the submittal of the final working drawings by contacting the Documents Unit at (916) 227-8252.

REVIEW OF WORKING DRAWINGS – STEEL STRUCTURES

Procedure:

The instructions in this Memo apply to working drawings for bridges or other major structures. Working drawings for railings, signs, miscellaneous metal, and other minor items are for the use of field personnel and are not routinely reviewed by the designer. See Article 10 for review of projects designed by Local Agencies and Consultants.

To provide uniform treatment in checking steel working drawings, the following procedures shall be followed:

1. The responsibility for checking working drawings is shared by the designer, the Structure Representative and Transportation Laboratory. Working drawings shall not be returned to the contractor until the designer has discussed and resolved the details with the other reviewers. The comments returned to the contractor must be acceptable to all reviewers.

   A brief memo shall be written by the designer to document controversial decisions or when it is necessary to keep other involved parties informed. For example, a memo is required for any changes or clarification of details in the contract plans. A copy of the memo is to be sent to the Structure Representative and two copies are to be sent to the Transportation Laboratory.

2. When the initial drawings, between six and ten sets, are received, the Documents Unit will forward two sets to the Transportation Laboratory, one set with correspondence to the Structure Representative and the reminder with correspondence will be sent to the design section involved. If less than six sets are received, the Documents Unit shall immediately request the missing sets. The Documents Unit will make this distribution.

   In the event drawings are received for review involving prestressing systems, one of the sets will be forwarded to the Chairperson of the Prestressing Committee for check enroute to the design section involved.

3. The Transportation Laboratory will make the sheets as required and return one set to design.

4. The set of drawings sent to the design section will be the work and file set, that is, it will be marked as necessary in yellow to indicate the checking performed, and in red to indicate any changes required.

5. One of the two sets of drawings sent to the Transportation Laboratory will ultimately be returned to the Contractor. It should not be stamped until all details are resolved between the Structure Representative, the Transportation Laboratory and the Designer and compatible comments transferred to the sheet.

Supersedes Memo to Designers 12-1 dated January 1982

6. Subsequent submittals of working drawings will not be routed out to the Structure Representative or the Transportation Laboratory. If there are any significant changes, the Designer will contact both groups and discuss them before the distribution of prints is made.

7. The Documents Unit will keep the latest set of drawings on file and make them available to the Designer as necessary. When corrected or revised drawings are received, the initial prints will be marked with blue and returned to the design section. All superseded sheets may be disposed of unless a claim or change order is anticipated and there may be a need to reconstruct the history of the project.

8. Members of the structural steel committee may be consulted at any time to assist with technical questions concerning shop practices and procedures.

9. Special Procedure for Structures Carrying Railroads

   a. Specifications require an initial submittal of ten sets of drawings. If less than ten are received, the Documents Unit shall immediately request the missing sets.

   b. From two to four sets are sent to the railroad for their review, with a request that they be *expedited*.

   c. At the same time one set is sent to the Structure Representative, two to the Transportation Laboratory, and the remainder to the design section involved.

   d. The design section is not to return an “approved” or “disapproved” set until all comments, including the railroad’s are received.

   e. When the railroad comments are received, the design section will mark the plans accordingly, or resolve differences as necessary.

   f. The set returned to the Contractor will incorporate both State and Railroad comments.

10. Review of projects designed by Local Agencies and Consultants

   a. Review and oversight of projects involving structures, designed and developed by local agencies or private consultants, is the responsibility of Local Assistance or the Externally Financed Projects Branch.

   b. Occasionally, others may be requested to review technical specialty areas such as walls, railings, earth retaining systems or projects having complex seismic concerns.

   c. Coordination and all plan distribution activities during the design phase will be handled by the Local Assistance or Externally Financed Projects Branch.

   d. Coordination of shop plan submittals by consultants will require special handling by the branch

involved.

11. Stamping of Working Drawings

a. Initial Review

1. If they are correct on initial review, the checker shall stamp and date.

Checker shall initial one set of prints only (yellowed check set). This set will be retained in

APPROVED
PURSUANT TO SECTION 5—1.02
OF THE STANDARD SPECIFICATIONS

JUL 7 1989

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES

the job file by the Documents Unit.

2. If any corrections *whatsoever* are noted, the sheets in error shall be returned for correction. The sheets with corrections shall be stamped and dated:

Checker shall initial one set of prints only. This set will be retained in the job file by the

Prints Reviewed by
State Division of Structures and
RETURNED

JUL 7 1989

FOR
CORRECTION

Documents Unit.

b. Second or subsequent review.

1. If stamped "Returned for Correction" on subsequent review, prints will be handled in the same manner as prints for initial review.

   If only a few minor corrections are made, all sheets needed for distribution must be marked with the same corrections and all stamped:

   Checker shall initial one set of prints only (yellowed check set). This set will be retained in

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DIVISION OF STRUCTURES

MINOR CORRECTIONS SHOWN
IN RED

   the job file by the Documents Unit.

12. Return to Contractor

    After checking and the discussion with all reviewing agencies is completed, all copies of the Working Drawings shall be returned to the Documents Unit. One of the stamped copies will be sent to the Contractor, and additional copies will be requested as needed for rechecking or for distribution.

13. Distribution of Final Approved Drawings

    a. Working drawing File – one print

    b. Contractor – one print

    c. Structure Representative – two prints (one for his use and one for Contractor's Field Representative).

    d. Transportation Laboratory – two prints (additional prints will be furnished where out-of-State

fabricators are involved).

e. Railroads – two prints for each Railroad company involved.

14. Final Disposition of Working Drawings

After completion of a project, the fabricator will furnish 35 mm film of working drawings to the Division of Structures as required by the specifications. These films are checked by the Documents Unit to verify that all required film has been received prior to sending them to file. The films are then filed in roll form by the Documents Unit.

After the films are received the file copies of Working Drawings will be sent to the responsible design section.

15. Guide for Checking Working Drawings

As a means of establishing uniform practice and avoiding omissions, but not as a substitute for common sense, the following outline is submitted as a general guide for checking Structural Steel Working Drawings:

a. Read the Standard Specifications and Special Provisions for the particular job. They may modify the usual procedure. Read the correspondence file; there may have been changes approved by the Office of Structure Construction since the contract was let. Call the Structure Representative to establish a working relationship, and to become familiar with any pending changes or special problems.

b. Changes from the contract plans or specifications, regardless of magnitude, should not be allowed unless they have been discussed and approved by the Structure Representative and Transportation Laboratory. Revisions may be satisfactory structurally but create administrative problems. Changes requiring Contract Change Orders as determined by the Structure Representative need special attention. These change orders could be grouped into two categories:

1. Those involving changes requested by the State and minor changes requested by the fabricator where there is no question on approval of the change order by both parties. The working drawings can be approved but the note “Contract Change Order to be processed” added to each detail sheet involved.

2. Those involving controversial changes requested by the fabricator. These should be returned to the fabricator with the note “Request must be made by the Contractor to the Resident Engineer for Contract Change Order.” The fabricator may ask that the working drawings be held by design pending such negotiation. Design should not hold any plans without such a request.

c. Review the Contractor’s erection procedure to be sure that it will satisfy the assumption for

continuity made in design. If the design assumptions are not met, the contractor must submit calculations for revised cambers and stresses. He may be required to increase plate thicknesses or change types of steel.

d. Check to see that all material shown in the working drawings conforms to the size, thickness and steel type shown on the contractor plans or with the requirements of an approved erection procedure.

e. The amount and method of camber should conform to the contract plans or with values computed to accommodate an approved erection procedure.

f. Check the size of all welds. If a welding sequence or procedure other than that shown is proposed, it should be reviewed by the Transportation Laboratory.

g. Check the direction of rolling of plates where specific orientation is required, and the location of butt splices and details of connections not dimensioned on the plans.

h. In general, check only those items listed above. For example, do not routinely make a detailed check of dimensions or the bill of materials.

Philip C Warriner

Guy D. Mancarti

RCA:jgf

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Volume II

BRIDGE CONSTRUCTION MEMO 175-0.0

SURVEYING

October 19, 1998

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RALPH P. SOMMARIVA, Chief
Office of Structure Construction

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Volume II

BRIDGE CONSTRUCTION MEMO 175-1.0

SURVEYING

October 19, 1998

GUIDELINES FOR BRIDGE CONSTRUCTION SURVEYING

Introduction

This memo summarizes Office of Structure Construction policy with respect to the extent of construction surveying to be performed by field personnel.

Although the primary purpose of this memo is to establish specific guidelines and thereby ensure uniformity in stakeout procedure, we recognize that no specific staking policy can be applied in all situations. Of necessity, surveying and stakeout techniques as well as the actual amount of surveying performed must be left to the discretion of the field engineer. To ensure statewide uniformity, however, all field personnel will be expected to conform as closely as possible to the policy as established herein.

Specification References

Section 5-1.07 of the Standard Specifications provides in part as follows:

> "Stakes or marks will be set by the Engineer as the Engineer determines to be necessary to establish the lines and grades required for the completion of the work specified in these specifications, on the plans and in the special provisions."

General Policy

As a general policy the Office of Structure Construction will interpret Section 5-1.07 of the Standard Specifications as provided herein.

Line and grade reference points will be set sufficiently close to the working area to enable the Contractor to accomplish their work using those tools which are normally associated with the work of a bridge construction crew. Such tools include string or wire lines, plumb bobs, carpenters' levels, 15 m (50') steel tapes, etc.

While it is recognized that a reasonable and prudent contractor should have surveying instruments on the job for their use in constructing the project, it is not required by our policy.

Specific Policy

Within our general policy, certain surveying procedures are subject to specific policy as further described herein. To ensure uniformity, procedures covered by specific policy must be followed on all projects.

1. Establish bent, abutment and wingwall reference points in accordance with the following:

   a Set a minimum of two reference points on each side of the footing.

   b Show distance to principal intersecting control line only. Do not reference intermediate points such as edge of footing, centerline of footing, etc.

   c Set inside reference points sufficiently close to the work that so contractor personnel in accordance with our general policy may use them.

   d Set elevation on inside reference point. Do not show cut to bottom of footing, top of footing, etc.

2. Do not stake the location of individual piles. Do not establish individual pile cutoff elevations.

3. Do not perform any survey work in connection with falsework construction.

4. Set elevation points at the bottom of walls, abutments and columns on the footings so that the Contractor is able to measure up and set their pour strips. Top of column, abutment and wall elevations should be provided to the contractor after they have been calculated and checked either on a grade sheet or on a profile plot with sufficient elevations for the contractor to set their pour strips.

5. Establish edge of deck line at 3 m to 6 m (10 to 20 foot) intervals on soffit forms with a transit. Establish sufficient grades at various places on the soffit forms to allow the Contractor to grade it with string lines or other similar means. Deck elevation points are to be set in accordance with Section 51-1.17, “Finishing Bridge Decks”, of the Standard Specifications.

6. Reference superstructure grades to top of deck only. Do not show cuts or fills to intermediate elevations.

7. Survey for control of barrier rail line and elevation should be performed in accordance with the following:

   a On straight bridges, establish line with a transit and set points at 3 m to 6 m (10 to 20 foot) intervals. On curved bridges, lines may be set from the edge of deck or by transit at the engineer's option.

   b On all bridges, take as-built deck elevations along (or near) rail centerline. Plot profile and make any necessary adjustments to correct for camber, deck irregularities, etc. Grades for fills, as necessary, should be provided to the contractor.

   c Final check should be by "eyeball" to ensure a pleasing appearance in the final product.

8. Bridge deck contour plans should be made available to the Contractor for their use.

Miscellaneous

The Standard Specifications require the contractor to carefully preserve the stakes and marks set by the engineer and make the contractor responsible for the cost of necessary replacement or restoration of stakes or marks which in the judgment of the engineer were carelessly or willfully destroyed by the contractor's operations. Any assessment of restaking charges should be done in accordance with District policies and procedures.

What the contractor can expect in the way of staking and how much notice is expected from them should be discussed at the pre-job meeting.

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180-6*	06/06/1997	Revised Column Casing Weld Backing Plate Thickness
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180-8.0	06/08/2012	Welding of Column Casings
180-9*	10/15/2002	Review of Contractor's Quality Control Plan (QCP) for Field Welding

Robert A. Stott, Deputy Division Chief
Division of Engineering Services
Structure Construction

**Denotes the document is a Bridge Construction Bulletin*

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BRIDGE CONSTRUCTION BULLETIN

Approved: _____
FOR **R. P. SOMMARIVA, Chief**
Office of Structure Construction

File: BCM 180-1.0
WELDING

Date: December 2, 1996
Expires: November 30, 1997
Supersedes: BCM 180-1.0 (3-30-87)
BCM 180-2.0 (3-30-87)
BCM 180-3.0 (330-87)

Subject: Field Welding of Structural Steel

There have been recent developments and problems associated with the workmanship on structural steel contracts and welding of structural steel members. This Bulletin provides an overview of the specificationrequirements, and provides a basic checklist of items to review before, during and after structural steel welding work. A contact list for welding support from the Materials Engineering and Testing Service (METS) is provided under Construction Bulletin, BCM 180-4.0, which is forthcoming within next month.

Work on Structural Steel members shall be governed by the requirements of the Contract Special Provisions, Standard Specifications, Section 55 "Structural Steel", Standard Specifications Section 75, "Miscellaneous Metal" and the most current publication of ANSI/AASHTO/AWS D1.5 "Bridge Welding Code" or ANSI/AASHTO/AWS D 1.1, "Structural Welding Code". The procedure for the handling of Structural Steel Working Drawings is found in Bridge Construction Memo 170-3.0.

Reference copies of Bridge Welding Code, AWSD1.5-95, and Structural Steel Welding Code, AWS D1.1-94, are available. Please contact your Supervisor.

Five attachments are included with this Bulletin: Attachment No. 1 is Bridge Welding Code, AWS D1.5, OSC Contract Administration Guide; Attachment No. 2 is Structural Welding Code, AWS D1.1, OSC Contract Administration Guide; Attachment No. 3 is Safe Code of Practices for Welding Inspection; Attachment No. 4 is Checklist for Compliance with Bridge Welding Code, AWS D1.5, Attachment No. 5 includes copies of Section III "Responsibilities of Caltrans Quality Assurance Person" and Appendix A "Field Inspection Procedure" born the METS Quality Assurance Manual.

Attachments

cc: All Jobs
ACM's
Senior BCE's
RBushey
PStolarski
BGauger

Bridge Welding Code AWS D1.5
OSC Contract Administration Guide

KNOWLEDGE

Review the requirements in the Special Provisions, the Contract Plans, and the Standard Specifications Section 55-l .02, "Welding".

Review the following sections of the Bridge Welding Code AWS D1.5:

- Chapter 3, "Workmanship"
- Chapter 4, "Technique"
- Chapter 5, "Qualification"
- Chapter 6, "Inspection"
- All other sections are to be reviewed as needed.

COORDINATION WITH METS

Prior to meeting with the Contractor at the start of the project, make arrangements to meet with a METS representative to discuss the contract and the welding requirements. This will help establish the roles and responsibilities regarding Caltrans quality assurance inspection.

Refer to the "METS Quality Assurance Manual" for clarification on roles and responsibilities of METS Quality Assurance (QA) personnel. (See Attachment No. 5)

Remember, the Structure Representative has the technical control over all structure work including welding. The Structure Representative is responsible to ensure that the welding is done properly, QA inspection is performed adequately, and the work is fully documented in the project files. METS personnel will provide assistance to OSC for QA inspection of welding work. However, due to limited staffing and difficulties in scheduling statewide METS inspection, OSC personnel must assist METS with QA inspection of welding.

NOTICE TO CONTRACTOR

At the preconstruction conference, schedule a pre-welding meeting with the Contractor. Suggest that they invite to the meeting, the appropriate Sub-Contractors (Welding, Fit up & Erection, Steel Fabricator, etc.) and the Quality Control Welding Inspector(s). Prior to the meeting, provide a copy of the meeting agenda to the contractor and suggest that they bring along their quality control plan, all welders qualification papers, testing information, and any other related welding documents.

Bridge Welding Code AWS Dl.5
OSC Contract Administration Guide

PRE-WELDING MEETING

Conduct the meeting with the Prime Contractor, Sub-Contractor(s), Contractor's Quality Control Welding Inspector and Materials Engineering and Testing Services (METS) representative to discuss the following:

Review specific welding requirements noted on the Contract Plans and Special Provisions.

Section 55-1.02 of the Standard Specifications requires the contractor to submit a "Quality Control Program" listing methods and personnel to satisfy the requirements of Part 6 of AWS Dl.5. Review the Contractor's quality control program (plan) and discuss any deficiency.

The Contractor is responsible for Quality Control (QC) and must appoint a Quality Control Welding Inspector(s). The Contractor's QC Inspector is responsible to review all of the welds and related work performed in the field and to verify that the work is in conformance with the approved Welding Procedure Specifications (WPS) on a continuous basis. Review the qualifications, and responsibilities for the contractors proposed QC Welding Inspectors. (AWS Dl.5, Part 6).

Review the requirements of the Welding Procedure Specifications (WPS), Procedure Qualification Record (PQR), Welder(s), Welding Operator and Tack Welder qualifications. (AWS Dl.5, Part 5, "Qualification"). (METS should cover this portion of discussion).

Suggest the Contractor submits a welding schedule identifying all contract welding work. This will assist field inspection of welds and allow proper lead time for identifying the proper weld test to be performed. Time frames for all test results submittals and-methods of reporting should be addressed.

Discuss AWS D1.5, Section 3.3 "Assembly", to review tolerance and proper positioning of members. Incorrect fit-up will normally result in deficient welds.

Review all documents to be submitted before during and after each portion of welding is performed. (Refer to Attachment No. 2, "Checklist for Compliance with Bridge Welding Code, AWS Dl.5 ")

Discuss corrective measures when welds are not in conformance with the contract documents. (AWS D1.5, section 3.7)

All conversations regarding welding should be documented in writing by the Structure Representative (SR) or SR Assistant. Include a list of all attendees.

Bridge Welding Code AWS **D1.5**
OSC Contract Administration Guide

FIELD OFFICE FILE

Documents related to welding work are to be filed in the contract's Project Record Files under the following categories:

Category 5 : Copies of all Correspondence
Category 37: Test results for all field tests
Category 41: Certifications for electrodes
Category 42: (New Category) Contractor's submittals (copies of approved Quality Control Program, approved WPS & PQR, welders qualifications, certified welder reports, nondestructive testing (NDT) qualification, certified test reports, calibration reports for NDT equipment, QC Inspector diaries, etc.)
Category 45: Structure Rep. Diaries (including meeting notes)
Category 46: Assistant Structure Rep. And METS diaries

The use of sub-categories should be utilized to keep the welding documents together.

INSPECTION

The Contractors Quality Control Inspector shall meet the inspection personnel qualifications as discussed in AWS D1.5, section 6.1.3.

The welding inspector from METS should be present on the first day of welding, unless other arrangements are made in the pre-welding meeting.

The Contractor's QC Inspector is responsible for all welding operations in the field and is required to monitor the fabrication, set up, root opening, groove angle, equipment settings, and welding papers, etc. on a regular basis. All procedures for welding, testing and documentation must be in accordance with the approved WPS. (AWS D1.5, section 6, "Inspection").

Check the welders welding conditions and verify that the welding tolerances are meeting the contract requirements established in AWS D1.5, Section 3.

The welders, weld operators, and tack welders shall meet the qualifications and use techniques that conform to the approved WPS (consult METS for assistance).

The surface preparation shall be cleaned as necessary to produce sound welds (AWS D1.5, Section 3).

Production rates should be monitored, as needed, to ensure WPS compliance. Production rate fluctuations, especially increases, may indicate non-conformance with the Specifications.

TESTING

Complete joint penetration groove welds in main members shall be QC tested by nondestructive testing. Personnel performing nondestructive testing shall be qualified in accordance with the American Society of Nondestructive Testing's (ASNT) Recommended Practice No. SNT-TC-lA, or equivalent (AWS Dl.5, Section 6.1.3.4). METS personnel will provide assistance for all testing matters.

Radiographic Testing (RT) shall be used for examination of complete joint penetration groove welds in butt joints subject to calculated tension or reversal of stress. See Contract Plans to identify the type of stress in members. If the stresses are not identified on the Contract Plan, contact the Designer.

Ultrasonic Testing shall be used for examination of all complete joint penetration groove welds in T- and comer joints. See Contract Plans to identify the type of stress in members. If the stresses are not identified on the Contract Plan, contact the Designer.

When required, RT and UT may be used to test all complete joint penetration groove welds in butt joints in compression or shear.

Requirements of RT and UT testing are found in AWS D1.5 Section 6.7.1.2.

Weld tabs (extension bars and run off plates) shall be removed prior to testing (AWS D3.12.2.)

Magnetic-particle Testing (MT) shall be used for examination of fillet welds and partial penetration groove welds joining primary components of main members (e.g. web to flange, diaphragm connection plates to web or flange, etc.). Magnetic-particle inspection of fillet welds is not required for secondary members. Consult with designer for verifications.

Requirements of magnetic-particle testing is found in AWS D1.5, section 6.7.2.

Per AWS D1.5 Section 6.5 "Inspection of Work and Records", The Contractors QC Inspector shall keep a record of all WPS qualifications or other tests that are made. The Engineer should get copies of all certified test results and place in the field office files.

Structural Welding Code AWS D1.1
OSC Contract Administration Guide

Similar steps mentioned in Attachment No. 1 for administration of contracts with AWS D1.5 code (Knowledge, Meeting with METS, Notice to Contractor, Pre-Welding Meeting, Filing, and Inspection) should be followed for contracts with AWS D1.1, Structural Welding Code requirements.

The contract requirements for welding, welder qualification, and inspection of welding for projects with AWS Dl.1 Code are similar to the projects with AWS D1.5 Code with the following exceptions:

- Currently there is no contract requirement for the Contractor to submit a "Written Quality Control Program". Future Specifications will be revised to include requirements for a program.

- The qualifications for a qualified engineer or technician proposed by the Contractor as the QC Welding Inspector, need to be "verified" not "accepted", by the Engineer.

- There is no mandatory nondestructive testing required by the AWS D1.1 Code. Welds can be accepted if the Quality Control Inspector and the METS welding inspector find the welding quality to be acceptable by visual inspection.

- When nondestructive testing other than visual inspection is specified in the Special Provisions, it shall be the contractor's responsibility to ensure that all specified welds meet the quality requirements of AWS Dl.l.

- If the Engineer or METS representatives has reason to believe that the welding quality does not meet the specifications, the contractor shall perform any requested testing or shall permit any testing to be performed by the State in conformance with AWS Section 6.

CODE OF SAFE PRACTICES
WELDING INSPECTION

1) All employees exposed to welding and weld inspection work must be trained in the hazards and precautions necessary to conduct the work safely.

2) Electrical Hazards
   - stay clear of welding leads, particularly in wet conditions
   - inspect leads for frays and missing insulation, all leads must be insulated
   - do not touch or remove the ground lead, unless directed to do so by the welder
   - welding equipment should be shut off when not in use

3) Fumes and gases.
   - Welding operations create harmful fumes and gases, position yourself upwind and away from the welding to avoid exposure
   - Do not enter confined spaces where welding is being done, unless properly trained and equipped as required by Caltrans Safety Manual Chapter 14
   - Be aware that welding on galvanized or paint coated steel (particularly lead paint) produces toxic fumes and smoke, stay away from these operations unless properly trained and equipped with respiratory protection (See Caltrans Safety Manual Chapter 15)

4) Eye Hazards
   - Never look directly or indirectly at welding work, unless you are wearing a welding helmet or goggles with lenses properly shaded for the type of welding being'done (generally a #14 shade is required for large electrodes). Be aware that reflected or indirect arc can also cause eye burns.
   - Welding operations should be isolated or shielded to prevent "flash" to adjacent workers or the traveling public.
   - Wear ANSI approved safety glasses on the job site to protect from flying particles.

5) Skin Protection

   - Stay clear of welding operations. Wear long sleeve shirts or coveralls to protect skin from ultraviolet rays generated from welding.
   - Be aware that metal parts may still be hot after welding is done. Wear gloves where appropriate.

6) Radiation

   - Weld inspection involves the use of radioactive sources, typically emitting gamma rays. These rays will penetrate clothing and skin, the best protection is to stay away. **Never touch or handle a radioactive source.** Contact the technician or inspection company immediately if you find a source out of it's storage container or unattended.
   - The inspection technician must establish an exclusion zone around the work, with warning signs and tape, based on expected and measured radiation emissions. **Do not enter this area unless properly trained and equipped with a radiation detector badge. The** maximum allowable exposure is 2 millirem/hour, but exposures should be kept as low as possible.

Checklist for Compliance with .dge Welding CODE, AWS D1.5

** The Engineer Shall verify these items meet the requirements of the project.

	Reference	If complete ( )
Identify welded connections as tension, compression, shear or reversal member	Contract Plans	
Identify all welds (full pen or partial pen)	Contract Plans	
Identify all members as main or secondary	Contract Plans	
Review special welding considerations	Special Provisions	
Review working drawings	Std. Spec. 55-1.02	
Review contractors written quality control program	Std. Spec. 55-1.02	
Review requirements of QC Inspection by the Contractor	D1.5 - 6.1.1.1	
Review requirements of QA Inspection by the Engineer	D1.5 - 6.1.1.2	
Engineer and METS approved Welding Procedure Specification (WPS) in file.	D1.5 - 5, Part A	
WPS Qualification - Gen. Requirements	D1.5 - 5.7	
Procedure Qualification Record per weld configuration	D1.5 - 5	
Welding Consumables (Electrodes)	D1.5 - 5.5	
Electrode certification - SMAW	D1.5 - 4.5.5	
Electrode certification - FCAW (s or g)	D1.5 - 4.12.3	
Electrode storage (sticks only)	D1.5 - 4.5	
Welder, Tack Welder, Welding Operator - General requirements and certification	D1.5 - 5.21	
Welding positions	D1.5 - 5.22	
Welder qualification test record	D1.5 - 5	
Prequalified Standard Joint	D1.5 - Fig 2.4	
Approved matching filler metals	D1.5 - Table 4.1, 4.2	
Preheat and Interpass temperature	D1.5 - 4.2	
Joint Fit-up tolerances	D1.5 - 3.3 Assembly	
Weld profiles	D1.5 - 3.6	
Backing bars or back gouging	D1.5 - 3.13	
WPS for repair work (non-conformance report)	D1.5 - 3.7	
Weld Termination and Cleaning of weld area	D1.5 - 3.11/3.12	
Technique / Procedures for SMAW	D1.5 - 4.6	
Technique / Procedures for FCAW (g or s)	D1.5 - 4.14	
Inspection of work and records	D1.5 - 6.5	
Obligations of the Contractor	D1.5 - 6.6	
Nondestructive testing (NDT)	D1.5 - 6.7	
All changes to be noted on Contract Plans (As-Built)		

**Note: Most of this information is found in the handout provided by METS.

SECTION III

Responsibilities of Caltrans Quality Assurance Person

1. Primary Responsibility

The primary responsibility of a QA inspector is to insure that the materials and workmanship provided by the contractor meet the requirements of the applicable specifications. The QA inspector is required to verify that all specifications, codes, and special provisions requirements are met and that the contractor's QC reports are in order. The QA inspector shall make random field inspections as a means to accomplish satisfactory QA confidence. Towards the achievement of this objective, the following activities shall be performed on a regular basis :

Schedule daily/weekly meeting times with contractor's QC personnel to monitor job progress and ensure contractors QCP is in effect.

Review all QC reports, weld documentation, and NDT certifications to ensure continued compliance.

Perform random review of radiographs to insure specification compliance. METS assistance at Sacramento is available.

Perform field verification inspections. A minimum of one inspection per location or one inspection per hundred field welds will be desirable.

Document all reported and discovered non-conformance issues and contractor's proposed solutions.

Generate QA inspection reports to be turned over to the engineer in the time frame agreed upon.

To assist the QA inspectors in the performance of the above functions, a ***Field Inspection Procedure*** with the relevant forms has been included in Appendix A . The QA inspector is required to fill out these forms, as applicable, during every field inspection, and disseminate the same to the engineer and all other impacted personnel.

2. Specification Issues.

2.1 It is not the function of the contractor nor the QA personnel to decide issues of materials or design. These issues have been previously decided by the specification, the design requirements, the referenced codes, or will be addressed

by the engineering staff as needed. Should a specification problem be brought to your attention, notify the engineer in writing as agreed.

2.2 The contractor must build the product as specified. The QA inspector insures that the product is built as specified by reviewing procedures, qualifications, technique, and documentation. An amount of field verification as determined by the METS Section Chief and the engineer is necessary. Any deviation from the requirements of the specification must be thoroughly investigated and approved in writing by the engineer, prior to implementation.

APPENDIX A

Field Inspection Procedure

For each inspection there will be a check list to assist the inspector in covering all the areas necessary to achieve a complete Quality Assurance program. The following is a list of forms.

1.) Spec. Review: This form will be used to make the Caltrans Engineer and the METS Inspector aware of all required specifications and Testing
A review of the contractors Quality Control and welding paper work.

2.) Prejob: This is a record of the meeting with the Prime contractor, Quality control, the Welding contractor, and Testing company.
A review of the contractors responsibility for Quality Control
A review of the Quality Control Inspectors duties and responsibilities.
A review of Nondestructive testing requirements and the necessary documentation.

3.) Daily Report: This a record of the METS Inspectors Quality assurance on the Job.
A review of the Quality control inspection reports.
A record of any interaction with Caltrans Personnel
A record of any interaction with the Contractors personnel
A record of any verification inspection and nonconformance.

4.) Nonconformance: This is a record of Unacceptable work.
Type of problem (Welding, Fitting, Procedural, etc.)
List of locations
How was the unacceptable work discovered.
Who was notified and when.
What is proposed to rectify the problem.
Each type of nonconformance shall be listed on a separate form.

5.) Notification: This form is documentation and notification to the Caltrans Engineer from the METS inspector that the work does not meet the Specification.
This form will be used when the contractors Q.C. has accepted or overlooked unacceptable work.
A request that the work in question be reinspected.
A request for a written explanation from the Q.C. inspector.

Quality Assurance for field welding

Caltrans Quality Control and specification review

Special Provisions
List all references to welding (Section and Paragraph) and brief description
Highlight all references to other Specifications

1._____

2._____

3._____

4._____

5._____

Standard Specifications
List all references to welding (Section and Paragraph) and brief description
Highlight all references to other Specifications

1._____

2._____

3._____

4._____

AWS (Welding and Nondestructive Testing)
Code: D-1.5 (Bridges)_____ **D-1.4** (Reinforcing) _____ **D-1.1(Structural)** ___

1.) **Obligations of the Contractor** (Section ___ Paragraph _____

2.) **Inspection Personnel Qualifications** (Section Paragraph_____)
Qualifications of Welding Inspector. (Review documentation)

3.) **Nondestructive Testing** (Section____ Paragraph _____)
Type of test required: RT___ UT____ MT _____
Extent of Testing (Section Paragraph _____)

4.) **Welding Procedure Specifications** (Section Paragraph ____)
(PQR) Procedure Qualification Record _____
(WPS) Welding Procedure Specifications _____

5.) **Welder Qualification** (Section ____ Paragraph ____)

Name	Tests Required	Positions

Quality Assurance for Field Welding

Welding and Inspection Q. C. / Q. A. Meeting

This meeting shall take place prior to the start of the job to assure the contractor will be in conformance with the specifications. **A** representative from the Prime contractor, the approved welding inspector, the Welding contractor, the personnel performing nondestructive testing, the Caltrans Engineer, and the METS inspector should be present.

All parties involved in in inspection or testing shall have copies of all required specification. This should be verified by asking each individual attending the meeting if they have (Special Provisions, Standard Specification, and AWS)

Prime Contractors Representative YES_____ NO_____
The Welding Inspector Q. C. YES_____ NO_____
The NDT Inspector YES_____ NO_____

The role of the **Prime Contractor** in assuring quality work is:

1. The contractor shall be responsible for visual inspection and nondestructive testing. The contractor is responsible for necessary correction of all deficiencies in materials and workmanship (Div.___ Section _____ Paragraph_____)
2. The contractor shall hire qualified and competent personnel to perform inspection and testing
3. The contractor shall schedule nondestructive testing to facilitate attendance by the QA Inspector when requested by the Engineer.

The role of the **Quality Control Inspector** is:

1. Review welding procedures and welder qualification to assure conformance to the specification.
2. Perform inspection prior to assembly, during assembly, during welding and after welding as specified in AWS and additionally as necessary to assure that materials and workmanship conform to the requirements of the contract.
3. The Inspector shall record the locations of inspected areas and the findings of all nondestructive tests, together with detailed descriptions of all repairs made.

The role of the **NDT Technician** is:

1. The Inspector shall identify with a distinguishing mark or adequate document control as approved by the Engineer all parts or joints that the technician tested and approved.
2. The technician shall perform nondestructive Testing in accordance with all applicable Specifications.
3. The technician shall approve satisfactory welds, or reject unsatisfactory welds and report the results to the contractor in writing the same day.

All parties attending meeting :

Print Name	Signature	Date

METS INSPECTOR FIELD REPORT

Contract #_____ Date_____

Job Name_____ Report #_____

Company Name, Address_____ CWI name_____

Arrival Time_____ Departure Time_____ CWI there?_____

Inspected CWI reports_____

Checked rod ovens____, Electrodes to specification_____, Weld procedures____, Welder Qual____

Joint fitup_____, Mill reports____,

Item Inspected_____ Location_____

Summary of discussions with Contractor/Caltrans personnel

Discrepancies noted

Corrections to be made by

Reported to RE

Signature

Quality Assurance for field Welding

Nonconformance Report

Contract Number _____ Date _____

Type of Problem: Welding_____ Fitt-up_____ Procedural_____ Other_____

Description _____

_____

_____

Location _____

_____

Who found the problem ? _____

_____

Who was notified and when ? _____

_____

Was the Caltrans Engineer notified ? _____

Name of the Quality Control Inspector _____

Was the Quality control Inspector aware of the problem ? _____

What is the Contractor proposing to correct the problem ? _____

_____

_____

_____

What is METS recommendation _____

_____

_____

_____

Print METS Inspectors Name _____

Title _____ Signature _____

Quality Assurance for Field Welding

Notification of Nonconformance

Contact Number. _____ **Date** _____

Description of nonconformance: _____

_____

_____

_____

Specification reference. Spec. _____ Section _____ Paragraph _____

Draw detail below:
Include dimensions, label areas, and highlight the problem areas with circles or arrows.

The METS Inspector shall notify the Caltrans Engineer of nonconformance as soon as possible. This report is for Caltrans Personnel only and used to help the Engineer assess the problem and the effectiveness of the contractors Quality Control. If it is determined that the Quality Control is not sufficient to assure that the materials and workmanship conform to the requirements of the Specification the Engineer shall request an explanation in writing from the Prime contractor. When unacceptable work is found by the Quality Assurance Inspector (METS) this should be considered evidence that the Quality Control is not sufficient. If it is determined that the Q.C. Inspector is not qualified based on the fact that he does not recognize unacceptable work the Engineer shall have the individual responsible removed and replaced.

METS Inspector _____ Signature _____

DIVISION OF STRUCTURE CONSTRUCTION *Bridge Construction Records and Procedures Manual*	B00-08
BRIDGE CONSTRUCTION BULLETIN **Approved:** _____ **R. P. SOMMARIVA, Chief** **Division of Structure Construction**	**File: BCM180-2.1** **WELDING** **Date: June 1, 2000** **Expires: None** **Supersedes: BCB 180-2 (12-16-96)**

Subject: Checklist for Welding Quality Control

For all projects advertised after April 1997 (and in some cases by addendum) a section entitled "Welding Quality Control" was added to the Special Provisions as well as a revision to Section 52-1.08 "Reinforcing". Additional revisions to both sections have occurred since that time.

The Welding Quality Control Section in the Special Provisions, supplements the following sections of the Standard Specifications: Section 49 "Piling," Section 52 " Reinforcement," Section 55 "Steel Structures," Section 56-1 "Overhead Sign Structures," Section 75-1.035 "Bridge Joint Restrainer Units," and Section 86-2.04 "Standards, Steel Pedestals and Post." Other Sections of the Standard Specifications will be supplemented by the welding quality control plan when required. The welding quality control section also addresses field and shop welding requirements.

Attached are checklists to assist you with understanding the requirements of the welding quality control section contained in the contract Special Provisions. These checklists include a list of applicable contract documents to review and an outline of the responsibilities of the Structure Representative, and personnel from the Division of Materials Engineering Testing Service, Office of Structure Materials (OSM) during each stage of welding. In addition to the checklist, there is a list of commonly used terms and definitions.

The Structure Representative is responsible for all welding. OSM personnel are available to provide advice, guidance, review the welding quality control plan, and perform field and shop welding QA inspection (refer to BCM 180-4 and BCM 180-9 for a list of OSM contacts and phone numbers).

Even though the checklists are extensive, you need to review your Contract Documents for the latest specification requirements.

Copies of the Structural Welding Code-Steel (AWS D1.1) have been assigned to every Area Construction Manager, and copies of the Structural Welding Code, Reinforcing Steel (AWS D1.4), and Bridge Welding Code (AWS D1.5) have been assigned to every Senior Bridge Engineer. **Additionally, the Special Provisions now require the contractor to provide the State, as part of their welding quality control plan, the applicable AWS welding codes for the applicable year noted in the Special Provisions.**

Attachments

C: BCR&P Manual Holders
Consultant Firms
PStolarski, OSM
BPieplow, Construction Program Manager

WELDING QUALITY CONTROL CHECKLIST

DEFINITIONS

The following is a list of definitions commonly used within the section "Welding Quality Control" of the Special Provisions and elsewhere in the contract documents. Additional definitions can be found in ANSI/AWS A3.0-94 "Standard Welding Terms and Definitions."

Certified Welding Inspector (CWI) for State Projects – Inspector certified in accordance with AWS QC1. For State projects the Quality Control Inspector will be a CWI.

FCAW – Flux Cored Arc Welding – An arc welding process utilizing a tubular electrode with the flux contained within the core. The electrode is supplied on a reel and is fed continuously to the welder's gun automatically.

FLUX – A material used to hinder or prevent the formation of oxides and other undesirable substances in molten metal and on solid metal surfaces, and to dissolve or otherwise facilitate the removal of such substances.

GMAW – Gas metal arc welding utilizes a bare or a flux-cored electrode. Gas from an external source is used for shielding. Normally a shop welding process. Often referred to as MIG welding.

Non-Conformance Report (NCR) – A written report originated by OSM which addresses a deficiency being performed and the contract documents not being fulfilled. The report will describe the problem, the location, the Quality Control Inspector response, the proposed solution, and OSM recommendation.

Non-Destructive Testing (NDT) – Testing or an inspection method which does not damage the element being tested (e.g. Radiographic (RT), Ultrasonic (UT), Visual (VT), Magnetic Particle (MT), Liquid Penetrant (PT).

Procedure Qualification Record (PQR) – Documentation indicating testing was performed to qualify a WPS.

Quality Assurance (QA) – This oversight is the prerogative of the Engineer and will be performed by a State representative.

Quality Assurance Inspector (QA Inspector) – The duly designated person who acts for and on behalf of the Engineer. This person is from OSM and will inspect the welding operation and write a welding report for the State.

Quality Control (QC) – Responsibility of the contractor. As a minimum, the Contractor shall perform inspection and testing prior to welding, during welding and after welding as specified in the contract documents and additionally as necessary to ensure that materials and workmanship conform to the requirements of the contract documents.

WELDING QUALITY CONTROL CHECKLIST

Quality Control Inspector (QC Inspector) – The person duly designated by the contractor, to perform inspection, testing, and address welding issues on the project. This person shall be responsible to the contractor for the quality control acceptance or rejection of materials, workmanship, and shall be currently certified as AWS Certified Welding Inspector (CWI) in conformance with the requirements in AWS QC1, "Standard and Guide for Qualification of Welding Inspectors."

Quality Control Manager (QCM) – A representative, employed by the prime contractor, who is responsible directly to the contractor for the quality of all field welding performed. This includes the materials and workmanship. The QCM reviews, approves, and submits all QC documents to the Engineer.

Quality Control Plan (QCP) or Welding Quality Control Plan (WQCP) – A plan submitted by the contractor to the State for each item of welding work to be performed. This plan contains all welding documents required by the contract (refer to the Special Provisions and QCP 1). No welding can begin until this plan is reviewed by OSM and approved by the Structure Representative.

QCP-1, QCP-5 and QCP-7 – These forms are used by OSM and the Structure Representative, as checklists to ensure the contractor's Quality Control Plan or Fracture Control Plan are complete.

SAW – An arc welding process utilizing a solid wire electrode that is fed automatically to the welding head from a reel. A granular flux is automatically deposited from a dispenser onto the molten weld deposit (normally a shop welding process).

Resistance Butt Welding (Flash Butt Welding) – A welding process in which the necessary heat is derived from an arc or a series of arcs established between the bars being welded prior to pressure being applied to join the ends together.

SMAW - Shielded Metal Arc Weld – An arc welding process utilizing a solid electrode with an outer flux coating.

Welding Procedure Specifications (WPS) – A document providing the required welding variables for a specific application to assure repeatability by properly trained welders and welding operators.

Welder's Qualification – Welders must be certified for type and position of weld and weld process. If not certified, tests can be performed to qualify the welders. Welders must be certified and approved by OSM before welding on State projects.

Welding Quality Control Plan (WQCP) – See QCP above.

WELDING QUALITY CONTROL CHECKLIST

PRIOR TO BEGINNING ANY WELDING WORK

The following contract documents should be reviewed before starting any welding work:

Specific References:

Standard Specifications Sections:
- Section 6-3.02, Testing by Contractor
- Sections 49, 52, 55, 56, 75 and 86 (as applicable to the work)

American Welding Society (AWS) - AWS D-1.1, D-1.4, D-1.5 (appropriate year)
- AWS D-1.1: Prequalification of WPS, Qualification, an Inspection
- AWS D-1.4: Direct Butt Joint Figure 3.2, Workmanship, Technique, Qualifications and Inspection
- AWS D-1.5: Figure 2.4, Workmanship, Technique, Qualification, Inspection, Welded Steel Bridge, and Fracture Control Plan

Contract Plans
Contract Special Provisions
Bridge Construction Records and Procedures Manual
- BCM 9-1.1
- Section 180 – Welding
- BCM 145-16

OSM forms QCP 1 & 5 – (attachment 4)

Before starting any welding, three items need to be completed. A pre-weld meeting with OSM personnel only, a pre-weld meeting with the contractor, and the review and approval of the Contractor's Welding Quality Control Plan (WQCP). These and other items are explained further below.

1. Inform OSM immediately after contract approval that welding, including shop welding, will be performed for your project (see BCM 180-4 & BCM 180-9 for a list of OSM contact phone numbers). At this time, set up an initial meeting with **OSM PERSONNEL ONLY** to discuss the welding requirements for the project, and to plan the pre-welding meeting with the contractor.

2. Conduct a pre-welding meeting with the prime contractor for each type of welding to be performed in the shop or in the field for the contract (i.e. piles, column casings, structural steel, reinforcing steel, miscellaneous metal, etc.). OSM will conduct this meeting if you so request. The Resident Engineer, Structure Representative, Prime contractor, QCM, QC Inspector, any welding subcontractor, suppliers or fabricators and the NDT firm should attend this meeting. The State should have their QA Inspector from OSM present to assist with the following discussion topics:

   a) The submittal and approval process for the WQCP. Supply and explain OSM form QCP-1 (attachment 4) to the contractor. Form QCP-1 is a checklist of the minimum requirements for the WQCP. The contract documents may require additional information to be submitted with the contractor's WQCP beyond those listed on form QCP-1.

WELDING QUALITY CONTROL CHECKLIST

PRIOR TO BEGINNING ANY WELDING WORK (Cont.)

b) Discuss the appropriate sections of the AWS code and contract documents as they pertain to the acceptance and approval process of the contractor's WQCP (a OSM welding inspector should cover this portion). The WQCP will be reviewed by OSM and must be approved by the Structure Representative prior to any welding in either the shop or the field. In order for the Structure Representative to accept the WQCP, personnel from OSM will have to review the contractor's WQCP. This acceptance of the WQCP may require OSM personnel to witness the welder(s) welding test plates and the testing of those plates before accepting the WQCP. The same is true of the PQR.

c) Remind the contractor to provide adequate notice prior to starting any welding work (request one week minimum advance notice). This will allow time to schedule an OSM QA Inspector.

d) Inform the prime contractor they are responsible for QC, and they must hire the QC Inspector (a CWI) and the NDT firm, unless stated otherwise in the Special Provisions (i.e. AISC Quality Certification Program, Category Cbr, Major Steel Bridges).

e) Discuss the frequency of inspection, visual and NDT, as well as the frequency of the QCM's submittal of the QC Inspector and NDT reports.

f) Establish a method to identify the welds and lot sizes. This needs to be established for traceability purposes.

g) Discuss the process to randomly select welds to be NDT (see BCM 145-16 for random selection method).

h) Discuss corrective measures when welding does not conform with AWS or the contract documents.

i) Discuss OSM agenda items an any additional requirements addressed in the contract documents.

j) Confirm all discussions of each pre-weld meeting in writing and send a copy to the contractor.

3. Obtain three copies of the contractor's WQCP. Using form QCP-1 and the contract documents, **review the contractor's WQCP and ensure the submittal is complete before forwarding to OSM - this will save review time**. Once the contractor's WQCP is complete, **send two copies** to OSM for their review. OSM will assist the Structure Representative with the acceptance and approval of the WQCP. Keep the other copy in your project files.

4. If resistance butt welding, or any other shop welding is to be performed, ensure OSM has approved the welding procedure, performed a shop audit if required, and performed any testing that may be required to accept the welding process (see BCM 165-10 "Ultimate Butt Splice").

5. After OSM has completed their review, they will notify you by phone, followed by an acceptance memo (QCP–5 for WQCP and or QCP-7 for the Fracture Control Plan, attachment 4) for your project files.

WELDING QUALITY CONTROL CHECKLIST

PRIOR TO BEGINNING ANY WELDING WORK (Cont.)

6. After reviewing OSM acceptance memo, and if it is acceptable, place the standard stamp 5-1.02, on both the WQCP and the approved WPS to be used on the project. **Do not place your PE number on the WQCP, the WPS, or the approval letter to the contractor.**

7. Send the contractor a letter approving their WQCP and request 7 copies of these approved documents.

PROJECT RECORDS FILES

WELDING DOCUMENTS ARE TO BE FILED IN CATEGORY 9.

NOTE: To limit duplication and confusion, the Structure Representative may want to use a cross-reference system with the other project record categories (ensuring records can be easily audited). For example, correspondences are filed in Category 5. If the issue is welding, the Structure Representative shall file the document in Category 9 as described in the Construction Manual Section 3-01-2 "Category 5, General Correspondence."

The following is a suggested list for filing welding documents.

1. The WQCP will be submitted for each item of work for which welding will be performed in the shop and field (i.e. piles, structural steel, rebar, etc.). The WQCP shall conform to the requirements of the Special Provisions and shall include, as a minimum, the items listed on form QCP-1. **Remember, welding is not allowed until the WQCP is accepted by OSM and approved by the Structure Representative.** Each approved copy of the contractor's WQCP is to be filed in Category 9 "Welding" along with forms QCP-1, QCP-5 and QCP-7 if required.

2. Structure Representatives and their Assistants shall file their reports/diaries in Category 45 and 46 respectively. If welding item work is included within the report, one of two things shall happen: write a separate report, or place a copy of the report in category 9. The welding report shall include: location and type of welding work, amount of production, welder, QC Inspector, QA Inspector, equipment, comments or observations made by either the QC or QA Inspectors and any other pertinent information.

3. OSM welding inspection reports are to be filed in Category 9. All others inspection reports from OSM should be filed in their appropriate Category as outlined in the Construction Manual.

4. If you receive an OSM Non-Conformance Report (NCR) it is to be filed in Category 9, along with the documentation showing what corrective action was taken (repairs and re-inspections of the non-conformance work). **It is the Structure Representative's responsibility to ensure the non-conformance work is corrected and additional testing and inspection is performed per the contract documents (OSM will only assist in the reinspection when requested and instructed as to the acceptance criteria).**

5. Contractor's QCM is to submit their welding report to the Engineer within 7 days following the performance of any welding. The Engineer shall review the report for completeness and ensure the welding was found to be satisfactory. The completed report is to be filed in Category 9.

WELDING QUALITY CONTROL CHECKLIST

PROJECT RECORD FILES (CONT.)

6. Copies of all welding correspondences are to be filed in Category 9 (reference Construction Manual 3-01-2).

7. Test results of all field and shop welding are to be filed in Category 9.

8. The contractor shall furnish to the Engineer a Certificate of Compliance for all welding and electrodes used, as required by the contract documents and in accordance with Section 6-1.07 "Certificates of Compliance" of the Standard Specification and Section 8 of the Special Provisions. These certificates shall be filed within Category 9 with reference to the appropriate section within the project files.

DURING WELD PRODUCTION

1. Make arrangements so an OSM welding inspector is present at the job site or the shop on the first day of welding (if the first day is not possible, then the next available day - the key is to provide OSM adequate notice). If welding is being performed in the shop, OSM should be informed by a "Notice of Materials To Be Used" (Form CEM 3101). This will ensure the welding for your project gets off to a good beginning and the QC Inspector has a clear understanding of the QA Inspector's role and expectations.

2. OSM is responsible for assisting the Structure Representative with QA inspection on the project. OSM is responsible for welding QA at the fabrication shop. Every effort shall be made to ensure a representative from OSM is present during production welding; however, if OSM is not available, the Structure Representative and/or the Assistant Structure Representative shall provide QA inspection and document their findings in their daily reports. The following items should be discussed with an OSM inspector before the pre-weld meeting with the contractor. This discussion should be done in the event a QA Inspector is not available during production welding.

   a) Verify the contractor is providing QC inspection and using the appropriate AWS code, Contract Special Provision, and Standard Specification to evaluate the weld and weld procedure. The contractor is to provide a sufficient number of QC Inspectors to perform the inspection prior to, during, and after welding. The inspection interval of each welder's work shall not lapse more than 30 minutes, as stipulated in the contract documents.

   b) Verify the welder is listed within the approved WQCP and is qualified and was accepted by OSM to perform the specified weld. For example, if the WPS calls for SMAW in the vertical position, make sure the welder is qualified to perform SMAW in the vertical position. Note: the AWS code specifically disallows a vertical downward progression of welding.

   c) Ensure the welders are following the approved WPS. Items easily verified, include: correct base metals, fit up, joint details (such as bevel angle and root opening), weld process, weld position, electrode type and size, travel speed, voltage and amp settings, preheat and interpass temperature, cleaning/slagging between each weld pass, number of weld passes, and ensuring the welder is placing a string bead and not a weave weld. The QC Inspector should also verify and record these items daily.

WELDING QUALITY CONTROL CHECKLIST

DURING WELD PRODUCTION (CONT.)

d) Review the appropriate AWS code to ensure welding **is not done** when the ambient temperature is too low, when surfaces are wet or exposed to wind, or when welders are exposed to inclement conditions (see the appropriate AWS Code under workmanship or technique).

e) Ensure backing plates, if shown in the WPS, are tight against the base metal or rebar (it might be necessary to grind down only the bar deformations that interferes with the tight fit, not the core area of the reinforcing steel). The Special Provisions, for bar reinforcement, requires the backing plate to be a flat plate. Backing plates are not to be removed for radiographing. If the backing plate is for welding a column casing refer to BCM 180-6 and BCM 180-7.

f) Ensure the electrodes are properly stored. For SMAW electrodes, once the hermetically sealed container is opened, or after electrodes are removed from drying or storage ovens, the electrode exposure to the atmosphere shall not exceed the times stated in the AWS code (typically 4 hours maximum). For FCAW electrodes, they shall be stored in clean and dry conditions at all times.

g) Ensure the welder does not make any errant arc strike (contact between the electrode and the base metal outside the weld area). If an errant strike does occur, the material is subject to rejection, but confer with OSM first.

h) Verify NDT and destructive testing, when required, is being performed properly an in accordance with the Special Provisions and other contract documents (OSM may be of assistance in this regard).

i) Keep an eye on the production and failure rate. A dramatic increase in production and a drop in the failure rate generally result in non-conformance with the WPS.

3. Obtain the QCM welding reports within 7 days or as specified by the contract documents, following performance of any welding. Review this report with the assistance of OSM to determine if the contractor is in conformance with their WQCP. Except for steel piling, this report must be reviewed and a written response approving or rejecting the report must be returned to the contractor within 7 days (your time frame may vary, read your Special Provisions). For piling, this review time will be specified in the Contract Special Provisions.

4. Review all reports regarding NDT, destructive testing, and radiographing. As described in the contract documents, all reports shall have the appropriate signature of the reviewer. For radiographs - the NDT technician, the person performing the review and the QCM shall sign these reports. The reviewers name shall be clearly printed or type written next to their signature. If they are not, return them to the QCM.

5. All radiographic envelopes shall have clearly written on the outside the names of the: QCM, NDT firm, radiographer, date, contract number, complete part description, and include the weld numbers or a report number as detailed in the WQCP. In addition, all innerleaves shall have clearly written on them the part description and include weld numbers, as detailed in the WQCP.

WELDING QUALITY CONTROL CHECKLIST

WELD ACCEPTANCE

1. There are different forms of NDT (VT, UT, RT, MT, and PT) that may be performed on weld elements, but the contractors QC Inspector will always perform a visual inspection (VT) and write up a daily report. The contractor is responsible to ensure all necessary and required NDT is performed. The contractor is also responsible to ensure all welding fulfills the requirement of the contract documents and the appropriate AWS codes. It is the Engineers prerogative to perform QA inspection. If the QC Inspector identifies a defect it is to be noted in the welding report along with the corrective action taken. If the QA Inspector identifies a defect, a Non-Conformance Report will be written and given to the Structure Representative that day**. These reports are not to be given to the contractor. It is the Structure's Representatives responsibility to notify the contractor in writing and ensure the defect is repaired and any additional testing is performed and evaluated. Inform OSM of the repair and request an inspection of the repaired weld.**

2. Welds can be accepted if both the Contractor's QC and OSM QA Inspectors find the welding quality to be acceptable by visual inspection and/or NDT, in accordance with the appropriate AWS code.

3. In addition to the inspection, the contractor shall furnish to the engineer, in accordance with Section 6-1.07 "Certificate of Compliance," of the Standard Specifications and Section 8 of the Special Provisions, a Certificate of Compliance for each item of work for which welding was performed. This certificate shall state that all of the materials and workmanship incorporated in the work, and all required tests and inspections of this work, have been performed in accordance with the details shown on the plans, and the requirements of the Standard Specifications and the Special Provisions.

PROJECT CLOSE OUT

1. The location of all splices need to be show on the as build drawings per BCM 9-1.1.

2. Met with the OSM representative to confirm all NCR and any other details are resolved before accepting the project.

REINFORCING STEEL CHECKLIST

PRIOR TO BEGINNING ANY WELDING WORK

The following items are in addition to those listing in Attachment 2 and are intended to assist you with the inspection of reinforcing steel welding on your project. Therefore, **before starting any welding, review the section entitled "Prior to Beginning any Welding Work" within the "Welding Quality Control Checklist" (attachment 2).**

Specific References:

Standard Specifications Sections:
- 52-1.08B, Butt Welded Splices
- 52-1.08D, Qualifications of Welding and Mechanical Splices

Structural Welding Code - Reinforcing Steel, AWS D1.4 (appropriate year)
- Welding Procedure Qualification
- Welder Qualification
- Direct Butt Joints - Figure 3.2
- Inspection

An approved WQCP is required before any welding is allowed. Review the requirements outlined in form QCP-1, contract documents and AWS D-1.4. AWS D 1.4 does not provide for prequalified welds; therefore, all WPS's and welders must be qualified by testing. You will need a copy of the PQR for each WPS that will be used on the project and the qualification test for the welder(s). The PQR and the welder qualification test must be witnessed by either a lab approved by OSM or by OSM personnel. This should be discussed at the meeting with the OSM representative and also related to the contractor at the pre-welding meeting.

PROJECT RECORDS FILES

In addition to those items listed in Attachment 2, the following items also need to be filed in Category 9 for reinforcing steel welding:

1. The contractor's QCM and OSM QA reports. These reports shall also include the following information when rebar welding and NDT is being performed:

   a) Evidence showing at least 25% of all butt welds were radiographed by the Contractor.

   b) Evidence the Contractor evaluated the results, corrected deficiencies, radiographed repaired welds and radiographed additional welds as required (review your Special Provisions for specific details on additional testing requirements when welds are rejected).

   c) If more than two repairs of any weld are required, the Contractor must submit a repair plan detailing the problem and their proposed solution. This will prevent excessive heat damage to the reinforcing steel in the vicinity of the weld (heat affected zone).

2. A summary sheet recording when radiographs were submitted to and reviewed by OSM personnel and the response to the contractor.

3. Test reports of destructive testing performed for resistance butt welds. OSM will review the testing and perform QA.

REINFORCING STEEL CHECKLIST

DURING WELD PRODUCTION

In addition to Attachment 2, the following items shall also be reviewed by the Structure Representative and/or their Assistant:

1. Preheat and interpass temperatures. Ensure the proper temperatures are being used for the grade of steel or Carbon Equivalent (CE) being used. Refer to the your Special Provisions, and AWS D-1.4-92 table 5.2 or contact OSM personnel.

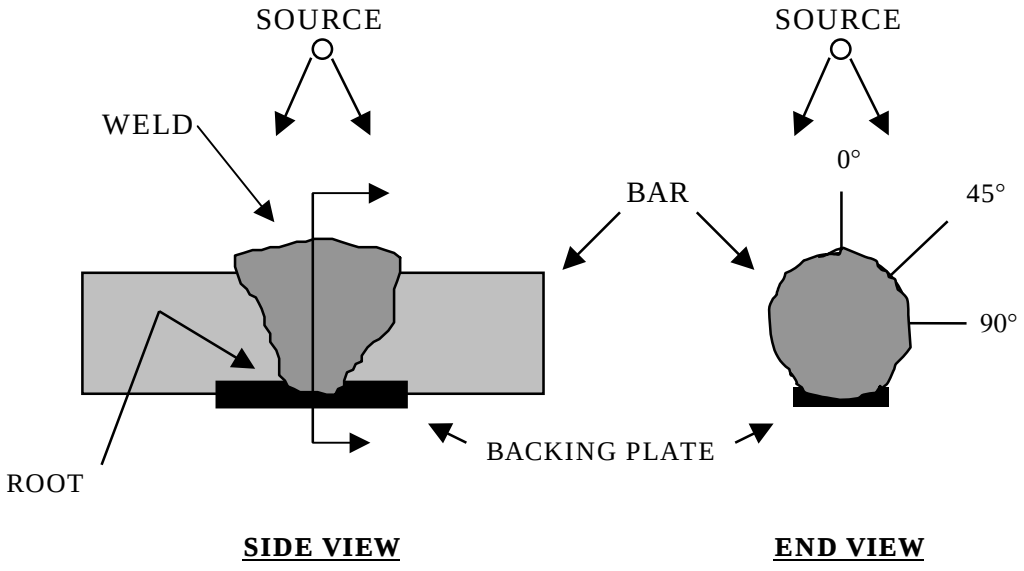
2. Bar alignment is within allowable tolerances. For example, AWS D-1.4-92 Section 4.2.3 states, for bars No. 10 or smaller the allowable offset is 1/8 inch. Additionally, Section 52-1.08 of the Standard Specifications states, the deviation in alignment of reinforcing bars at a welded splice shall not be more than 1/4 inch over a 3-1/2 foot length of bar.

3. When specified, a minimum of 6 inches on either side of the welded splice is covered with an insulated wrapping to control the rate of cooling after welding is complete. The method of protecting the weld area from heat loss shall be addressed in the approved WQCP.

4. Randomly select welds to be radiographed (for the random selection process, see BCM 145-16). Verify radiographs are being taken on at least 25% of the randomly selected production lot. If welds or radiographs are rejected, verify additional welds are being radiographed and re-shots of the repaired welds are taken in accordance with the Special Provisions.

5. Verify tests are being performed properly in accordance with the Special Provisions and other contract documents (assistance from OSM is required). Radiographs are to be taken at zero degrees from the top of the weld and perpendicular to the root of the weld as shown below.

TOP



REINFORCING STEEL CHECKLIST

WELD ACCEPTANCE

The Contractor should be encouraged to submit radiographs in a timely manner. This will allow the Contractor the opportunity to make corrections if necessary before the work progresses too far. Since the quality of the welding and the radiographing is the responsibility Contractor, the Contractor may choose to continue the work without waiting for OSM review and comment. If so, the contractor proceeds at his own risk and should be informed in writing.

The following items and those in Attachment 2 need to be obtained before accepting any reinforcing steel welding.

1. The contractor shall evaluate the radiographic film and the weld for acceptability and make any necessary repair to the weld and perform additional testing per the contract documents if required.

2. All radiographs, approved, reshot and/or rejected by the QC Inspector, must be reviewed by OSM. When the film is delivered to the Structure Representative, the Structure Representative should prepare a cover memo (attach to the radiograph film) requesting a review by OSM personnel. Before sending your memo and the film, check with your local OSM office for direction and proper sending instructions. On the memo, please include the Structure Representative's name, telephone number and fax number. Each piece of film shall include the contractor's name, date of radiograph, name of NDT firm, initials of the radiographer, contract number, part number, and weld number. The letter "R" and repair number shall be placed directly after the weld number to designate a radiograph of a repaired weld.

3. OSM personnel will review radiographs submitted by the contractor and phone the results to the Structure Representative within seven (7) calendar days after the review or as stated in the contract documents. A written report will follow within 10 working days. To ensure a complete review of the contractor's QC inspection, the radiographs of welds rejected by the contractor's QC Inspector will be reviewed by OSM. OSM will report their findings or the rejections to the Structure Representative as <u>information only</u>. These findings will be reported as either:

   a) "Reviewed film and interpretations submitted by the Contractor are consistent with the Office of Structural Materials findings."

   Or

   b) "Reviewed film and interpretations submitted by the Contractor are inconsistent with the Office of Structural Materials findings and we recommend the Contractor review the QC procedures currently in use."

4. The Structure Representative can accept welds if both the QC Inspector and the QA Inspector agree the welding quality is acceptable by visual inspection and NDT in accordance with AWS D1.4.

REINFORCING STEEL CHECKLIST

WELD ACCEPTANCE (Cont.)

5. In addition to the inspection, the contractor shall furnish to the Engineer, in accordance with Section 6-1.07 "Certificate of Compliance," of the Standard Specifications and Section 8 of the Special Provisions, a Certificate of Compliance for each item of work for which welding was performed. This certificate shall state that all of the materials and workmanship incorporated in the work, and all required tests and inspections of this work, have been performed in accordance with the details shown on the plans, and the requirements of the Standard Specifications and the Special Provisions.

PROJECT CLOSE OUT

1. The location of all splices need to be show on the as build drawings per BCM 9-1.1.

2. Met with the OSM representative to confirm all NCR and any other details are resolved before accepting the project.

CONTRACTOR'S QCP SUBMITTAL FOR WELDING

To: _____, Resident Engineer **Date of this Submittal:** _____,
Tel. No.: _____, **Fax No.:** _____,

From: _____, **Contract No.:** _____,
Welding Firm: _____ **NDT Firm:** _____,

Materials to be Welded: ☐Struct. Steel ☐Misc. ☐Rebar ☐Col. Casings ☐H-Piles ☐Pipe Piles
NDT Required : ☐RT ☐UT ☐MT ☐Visual Only
Specifications : ☐D1.1 (yr) ☐D1.4 (yr) ☐D1.5 (yr) ☐D1.6 (yr)

Our Quality Control Plan for welding to be performed on the subject contract is submitted for your review and approval.
The items checked below are submitted herewith.

QCP ITEMS TO BE SUBMITTED AS A MINIMUM	SUBMITTED	N/A	For R.E. Use
1. Organization Chart showing all QC Personnel & their duties.			
2. Name & Qualifications of Quality Control Manager (QCM).			
3. QC Inspectors - Provide copy of current AWS CWI Certification and eye exam for each Inspector to be used in the work # of submittals____,			
4. Names & Qualifications of Asst. QC Inspectors – Provide current AWS Assoc. CWI Certification or resume. # of submittals_____,			
5. Procedure Manual of NDT firm: certified personnel, NDT equipment, test procedures, calibration methods, methods and frequencies of tests, safety procedures and report forms to be used.			
6. Certifications for Level II NDT Technicians. # of submittals____,			
7. Methods and frequencies of NDT Inspections.			
8. List of Visual Insp. Tools (weld gages, tempstiks, lights, etc.).			
9. Procedures frequencies and extent of Visual Inspections.			
10. Method of tracking and identifying weld joints & welders.			
11. Daily Production & Inspection Log of Welds by QC Inspector.			
12. Action Plan for reporting non-conforming welds.			
13. Prequalified WPS (PQR not required). # of submittals____,			
14. WPS' requiring PQR testing. (PQR tests Must be State Witnessed) # of submittals & tests____,			
15. Standard Weld Repair Procedures.			
16. Electrode & Shielding Gas Certs. for each weld process. # of submittals____,			
17. Welder Qualifications for each process & position that each welder will perform. (Must be State Witnessed) # of submittals____,			
18. Sample Certificate of Compliance form to be used.			
19. One copy each of applicable AWS Welding Codes			
20. Other			

QCM Signature:_____ _____ (Printed Name)

METS LETTER OF TRANSMITTAL

REVIEW OF CONTRACTOR'S WELDING QUALITY CONTROL PLAN

To: _____, Resident Engineer
Tel. No.: _____

Date of this Transmittal:
Fax No.:

From: _____, Branch Chief
Date of Receipt (R.E.):_____

Contract No.:
Date of Receipt (METS):

The Contractor's Quality Control Plan Submittal #:_____**Rev. #** _____ **has been reviewed.**

☐QCP substantially **complies** with specification requirements and approval is recommended.
☐QCP **needs to be resubmitted** and unacceptable (reject) items corrected as per comments.
(See attached QCP5-NC for Non-Conforming Item Comments)

General Contractor: _____
Welding Firm: _____

Contractor's QCM:
NDT Firm:

Materials to be Welded: ☐Struct. Steel ☐Misc. ☐Rebar ☐Col. Casings ☐H-Piles ☐Pipe Piles
NDT Required : ☐RT ☐UT ☐MT ☐Visual Only
Specifications : ☐D1.1 (yr) ☐D1.4 (yr) ☐D1.5 (yr) ☐D1.6 (yr)

QCP ITEMS REVIEWED	COMPLIES	DOESN' T COMPLY	N/A
1. Organization Chart showing all QC Personnel & their duties			
2. Name & Qualifications of Quality Control Manager (QCM)			
3. QC Inspector-AWS CWI Certification and eye exam for each. # of submittals			
4. ASST QC Inspector - AWS CAWI Certification for each. # of submittals			
5. Procedure Manual of NDT firm: certified personnel, NDT equipment, test procedures, calibration methods, methods and frequencies of tests, safety procedures and report forms to be used.			
6. Certifications for Level II NDT Technicians # of submittals			
7. Methods and frequencies and extent of NDT Inspections			
8. List of Visual Insp. Tools (weld gages, tempstiks, lights, etc.)			
9. Procedures frequencies and extent of Visual Inspections			
10. Describe method of tracking and identifying weld joints and welders production			
11. Daily Production & Inspection Log for Welds for use by QC Inspector			
12. Action Plan for reporting non-conforming welds			
13. Prequalified WPS (PQR not required) # of submittals			
14. WPS' requiring PQR tests. # of submittals			
15 Standard Weld Repair Procedures			
16. Electrode & Shielding Gas Certifications for each process # of submittals			
17. Welder Qualifications # of submittals			
18. Sample Certificate of Compliance form to be used			
19. Other :			

METS REVIEWER:_____
Form METS QCP-5 rev. 4

Date Review Completed:
file: Loc: XX.20. A or B

CONTRACTOR'S FRACTURE CONTROL PLAN (FCP) SUBMITTAL FOR WELDING

To: _____,Resident Engineer
Tel. No.: _____

Date of this Transmittal:
Fax No.:

From: _____, Branch Chief
Date of Receipt (R.E.):_____

Contract No.:
Date of Receipt (METS):

The Contractor's Fracture Control Plan Submittal #:_____**Rev. #** _____ **has been reviewed.**

☐FCP substantially **complies** with specification requirements and approval is recommended.

☐FCP **needs to be resubmitted** and unacceptable (reject) items corrected as per comments.

(See attached QCP7-NC for Non-Conforming Item Comments)

General Contractor: _____
Welding Firm: _____

Contractor's QCM:
NDT Firm:

Specification: D1.5 (yr)

FRACTURE CONTROL PLAN (FCP) ITEMS TO BE SUBMITTED AS A MINIMUM	COMPLIES	DOESN' T COMPLY	N/A
1. Base Metal used meet the project and code requirements			
2. Consumable meet the requirements of heat or lot testing by the manufacturer			
3. Weld metal strength and ductility conform to tables 4.1 or 4.2			
4. Weld metal toughness meets table 12.1 requirements or the undermatching yield strength of a minimum toughness of 25ft-lb @ -20°F.			
5. WPS' requiring PQR test (according to section 12.7). # of submittals:			
6. Prequalified WPS (PQR not required) (According to section 12.7.1) # of submittals:			
7. Base metal repair procedure			
8. Tack weld procedures (According to section 12.13). # of submittals:			
9. Lead QC name, qualifications, and resume. Work history needs to show a minimum of 3 years experience in steel bridge fabrication inspection.			
10. NDT methods, personnel qualifications, eye exams, frequency of testing, reports to be used, and written practice of NDT firm. (see Sec. 12.16.1.2-5)			
11. Electrode & Shielding Gas Certs. for each weld process and base metal combination. # of submittals:			
12. Welder Qualifications for each process & position that each welder will perform. Welder Qualification tests shall be within 6 months of FCM work and shall be qualified by both mechanical (bend) and radiograph tests according to section 12.8.2 and 5 Part B. # of submittals:			
13. Daily Production & Inspection Log of Welds by Lead QC Inspector			
14. Noncritical Repair Welding Procedures such as surface discontinuities. Nocritical repair WPS shall meet the requirements of Section 12.17.2 and 12.17.2.1			
15. Critical Repair Welding Procedures (According to Section 12.17.3).			
16. Other:			

METS REVIEWER:_____

Date Review Completed:

Form METS QCP-7 FCP rev. 4

File Location: XX.20.A or B

BRIDGE CONSTRUCTION BULLETIN **Approved:** _____ **R. P. SOMMARIVA, Chief** **Office of Structure Construction**	**File: BCM180-3** **WELDING** **Date: December 16, 1996** **Expires:** **Supersedes: AJ96.14, AJ96.13**

Subject: Reinforcing Steel Welding and Testing

Pursuant to Section 6-3.02, Testing By Contractor, of the Standard Specifications, it is the Contractor's duty to conduct tests in accordance with procedures which are carefully planned, managed and documented by qualified personnel who are specifically designated to perform these functions. In accordance with Section 52-1.11, Payment, of the Standard Specifications full compensation for furnishing and testing sample splices, for radiographic examinations performed by the Contractor, and for furnishing access facilities for inspection and nondestructive testing by the Engineer shall be considered as included in the contract price paid per pound for the bar reinforcing steel involved and no additional compensation will be allowed therefor.

For going projects that have welding and have not received information from the contractor regarding the welding of reinforcing steel, the following procedures should be followed:

- The Resident Engineer is to send a letter to the contractor ordering a submittal of information detailing the contractor's plan to control quality of field welding of bar reinforcement. Use the draft letter in Attachment 1 to this bulletin so that all contractors are treated consistently.

- The Resident Engineer is to review the contractor's submittal for completeness and compliance with the Specifications.

- If the required information is not received within five working days of receipt of a request, the Resident Engineer is directed to temporarily suspend all field welding of bar reinforcement due to the contractor's failure to carry out an order to submit information and failure to demonstrate control of the workmanship, pursuant to Section 8-1.05, Temporary Suspension of Work, of the Standard Specifications.

For projects where the contractor has not started work, the information within Attachment 1 should be received from and discussed with the contractor during the pre-job meeting.

Attachment

cc: StrReps
BCEs
ACMs
OSC HQ Staff
Consultant Firms
Pstolarski, METS
BGauger, Construction Program Manager

From Hither to Yon
13-PAC-00-1.0/10.0
13-123454

June 28, 1996

Contractor, President
Construction Company Name
Company address
Company City, State, Zip Code

Dear Contractor:

Pursuant to Sections 6-3.02 of the Standard Specifications, submit the following, in writing, within five working days of receipt of this order.

1) Contractors' quality control plan for field welding of bar reinforcement as required by Section 52-1.08, "Splicing," and Section B-1.01, "Subcontracting," of the Standard Specifications, American Welding Society (AWS) D1.4, and the special provisions. The minimum requirements are as follows:

   a) List of names and qualifications of the Contractors' Quality control manager and all of the nondestructive testers, welding inspectors, and welders who perform and control the field welding of bar reinforcement work.

   b) Welder qualifications shall include all tests performed to qualify the welders, and verification that the tests were witnessed by the Engineer.

   c) The Contractors' field welding procedure specification for bar reinforcement, including documentation of the tests performed to qualify the specification, and verification that the tests were witnessed by the Engineer.

   d) Description of how the Contractors' quality control plan is updated when any personnel or procedures change.

   e) Description of methods for identifying, tracking, and reporting on; all field welding; quality of the material and workmanship including list of electrode classification, position, type of welding current and polarity; visual inspections; destructive and non-destructive testing; repairing of any deficient welds and reinspection of all repaired welds.

   f) Description of the methods used to ensure the quality control methods are being followed including the frequency of sampling and testing and frequency of reports or submittals to the Contractor.

2) Name of the Contractors' designated person who the Engineer can henceforth consider as the Contractors' quality control manager (QCM). The QCM shall be directly responsible to the Contractor. The QCM shall not be a currently employed subcontractor or currently employed by a subcontractor.

   a) The QCM shall be responsible for administration of the Contractors' quality control plan for field welding of bar reinforcement including but not limited to, the following:

- Review of welding and test records and,
- Assuring compliance with the Contractor's quality control plan for field welding of bar reinforcement and,
- Advising the Contractor and the Engineer of all deficiencies in the welds, tests, inspections and quality control plan for field welding of bar reinforcement compliance.

b) The QCM shall be the individual responsible for submitting and receiving all quality control correspondence and required submittals regarding the field welding of bar reinforcement to and from the Engineer. All such submittals shall be reviewed and signed by the QCM prior to submittal to the Engineer.

. If the plan is not received within the five working day period or if the plan is determined by the Engineer to not satisfy the minimum requirements stated above, field welding of bar reinforcement shall be temporarily suspended, pursuant to Section 8-1.05 of the Standard Specification by reason of the Contractor's failure to carry out an order to submit information and for failure to demonstrate control of the workmanship.

In addition, the QCM shall submit daily log summaries of welding, testing, and inspection for field welding of bar reinforcement to the Engineer at the end of every five working days of field welding of bar reinforcement performed.

Sincerely,

Resident Engineer

ax Prime Contractor, Home Office
Construction Program Manager, Sacramento
District Division Chief - Construction
R.E. File

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 180-4 WELDING**
Approved: Original Signed by Dolores Valls **Dolores Valls, Deputy Division Chief** **Offices of Structure Construction**	**Date: October 15, 2002** **Expires: July 1, 2003** **Supersedes: BCM 180-4 Dated August 1, 1997**

Subject: Welding Support from METS

The following link provides contacts within the Office of Materials Engineering and Testing Services (METS) for various welding questions, testing, and/or inspection. Utilize the Structure Material Representative appropriate to your EA or District. In addition to this link, it is recommended that other Structure Representatives who have worked with similar conditions or materials be contacted to ease the burden on lab personnel.

http://www.dot.ca.gov/hq/esc/Translab/smforms/StructuralMaterialsRepresentatives.pdf

cc: BCR&P Manual Holders
Consultant Firms
R. Pieplow, HQ Const.
P. Stolarski, METS

BRIDGE CONSTRUCTION BULLETIN **Approved:** R P Sommariva **R. P. SOMMARIVA, Chief** **Office of Structure Construction**	**File: BCM 180-5** **WELDING** **Date: December 1, 1996** **Expires: None** **Supersedes: AJ96.12**

Subject: Welding Support from METS for Projects Requiring Radiographs

If you currently have readiographs for a project, send the readiographs to the Sacramento office of Materials Engineering and Testing Services (METS) with a cover letter showing the Contract number, the structure name/number, what part of the structure are the welds located, and a person to contact once the radiographs have been reanalyzed by the Sacramento METS Lab. Send a copy of this memo to the OSC office in Sacramento.

For new radiographs, Sacramento METS (Materials Engineering and Testing Services) has issued guidelines to their employees in the Los Angeles and Emeryville labs. Below is an excerpt from their memo:

1. "Try to attend the pre-construction meeting between the contractor and Caltrans. This will provide the opportunity to go over what is expected during the welding phase of the work.

2. Upon notification by the Structure Representative that welding is ready to begin, visit the jobsite to review the welding procedures, fit-up etc., as well as provide assistance to the jobsite inspector as to what to watch for during the welding operation.

3. If radiographs are to be taken, spot check the bars by the Structure Representative to be shot to make sure they are visually acceptable. This can be done during the day, even if the actual shots will be taken at night.

4. If unable to witness a portion of the actual radiographs being taken, instruct the Structure Representative to have the film delivered to his or her office by the radiographers company at the start of the next day.

5. Visit the Structure Representatives office the next day and review the film for the following:

A. Film identification
- -Contract Number
- -Location (bent, column footing, etc.)
- -Weld Number verified by Q.C.
- -Technicians I.D.
- -Make sure information on envelop matches what is in the package

B. Film Quality
- -Density (2.5 to 3.5)
- -Check density in weld area
- -Check density in penetrameter

-Check to see if film is free of artifacts

C. Penetrameters
-Size denoted by number
-Placement/Location
-Thickness of shim when required
-Clearly identify hole as required (2T-4T)

After review of the film for items above, send (or have Structure Representative send) the film to Sacramento for review of welds for acceptance or rejection per Office of Structure Construction memo of May 8, 1996 (AJ96.10), (Structural Materials Branch, 5900 Folsom Blvd., Sacramento, CA 95819 ATTN: Paul Hartbower or Frank Reed.)

At the time of the initial review of the film by our field inspectors, the inspector SHOULD NOT accept or reject the weld. However the inspector SHOULD accept or reject the quality of the film.

Many of the problems we have experienced in the past with regard to welding inspection hopefully will be resolved once we get the additional training and staff in place. I feel we need to remember that we are providing a service and our client, in this case, is Structure Construction."

cc: StrReps
BCEs
ACMs
OSC HQ Staff
consultant Staff
RBushey, METS
BGauger, Construction Program Manager

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 180-6 WELDING**
Approved: FOR **R. P. SOMMARIVA, Chief Office of Structure Construction**	**Date: 6/06/97 Expires: 6/06/98 Supersedes: None**

Subject: Revise Column Casing Weld Backing Plate Thickness

The attached memorandum (Attachment 1) recommends that the thickness of the weld backing plate for column casings not exceed 3/8". Currently, most design details require backing plates to be the same thickness as the casing up to 1/2" thick. If the contractor requests to use a backing plate with a thickness less than the thickness of the column casing, then it is permitted that an appropriate no cost/no credit Contract Change Order be issued to allow:

- backing plates for column casing to have a thickness equal to 3/8".

Attachments

c: BCR&P Manual Holders
PStolarski, METS
RBushey, METS
ERDavisson, Chief Structure Design
BGauger, HQ Construction Program Manager
Consultant Firms

Business Transportation and Housing Agency

Memorandum

To MR. RICHARD SPRING
MS. DARCY HASSLER
MR. STEVE ELLIS
DMEs - Dist. 1,2,3,6,10,11

Date April 4, 1997

File Policy, Materials, Inspection Guidelines

Fom **DEPARTMENT OF TRANSPORTATION**
ENGINEERING SERVICE CENTER
Office of Materials Engineering and Testing Services, MS #5

Subject: Column Casing Inspection/Weld Backing

Currently, a general note in most of the design drawings require "backing plates to be the same thickness as casings up to 1/2" thick" and in some cases a statement that "backing must match casing thickness" with no limit on that thickness.

AWS D1.5-1995 Section 3.13.3 suggests the following backing thickness based on the welding process used: SMAW-3/16", GMAW-1/4", FCAW self shielded-l/4", FCAW gas shielded-3/8" and SAW-3/8".

There is no structural advantage to require backing in excess of 3/8" and fabricators have struggled to break the 1/2"+ backing material sufficiently to allow for proper field fit-up of casing components.

In the interest of a more consistent and better quality product, I recommend that weld backing not exceed 3/8". This should be effective immediately and enforced on all current and future casing projects.

Philip J Stolarski

PHILIP J. STOLARSKI. Chief
Structural Materials Branch

PJS/mdb

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 180-7 WELDING**
Approved: R. Sommariva **R. P. SOMMARIVA, Chief** **Office of Structure Construction**	**Date: 7/15/97** **Expires: N/A** **Supersedes: None**

Subject: Inspection of Weld Backing Plate for Column Casing

Attached is a memorandum from Roy Bushey to Ralph Sommariva, referring to the memorandum, dated January 25, 1995 from Richard Spring to METS field inspectors addressing the continuous/non-continuous welding of backing bars used in conjunction with column casings.

After further discussion with METS, the following clarifications were obtained:

> When welding column casing sections together in the manufacturer's shop, the backing bars are to have a continuous full length weld.
>
> The backing bars used to weld the column casing sections together in the field will only be welded to the column casing on one side by non-continuous welds. These welds will be 2" long and at 8" centers. This backing bar does not need a continuous full length weld.

Attachment

c: BCR&P Manual Holders
RBushey, METS
BGauger, HQ Construction Program Manager
Consultant Firms

State of California

Business, Transportation and Housing Agency

Memorandum

To: MR. RALPH SOMMARIVA, Chief
Office of Structure Construction

Date: June 6, 1997

File: Policy, Materials, Inspection Guidelines

From: **DEPARTMENT OF TRANSPORTATION**
ENGINEERING SERVICE CENTER
Office of Materials Engineering and Testing Services - MS #5

RECEIVED
JUN 1 0 1997
BY _____

Subject: Column Casing Inspection

It has come to our attention that some of the Structure Construction personnel have questioned the noncontinuous weld on the backing bars used in conjunction with column casings. Back in 1995, the fabricators were having problems with distortion of the edge of the column casings due to the heat generated by the continuous weld. In as much as there was no real need to have the backing bar welded continuously in order to produce a satisfactory field weld, we allowed the fabricator to make the weld as stated in the attached memo dated January 25, 1995.

This memo also addressed some other concerns which were present at that time. The main purpose was to maintain consistency in our overall inspection process.

If you could make this information available to your construction personnel, it may help to relieve some of the questions.

ROY BUSHEY, Chief
Office of Materials Engineering
and Testing Services

Attachment

cc: PStolarski
RSpring
DHassler
FReed
SEllis
DJones
SMFiles

RJW/llb

State of California

Business, Transportation and Housing Agency

Memorandum

To: MR. HERNANDO MORALES
MR. LARRY WEBSTER
DMEs - Dist. 1,2,3,4,5,6,9,10,11

Date: January 25, 1995

File: Policy, Materials, Inspection Guidelines

From : DEPARTMENT OF TRANSPORTATION
ENGINEERING SERVICE CENTER
Office of Materials Engineering and Testing Services

Subject : Column Casing Inspection

Due to some confusion and inconsistency, in the interpretation of column casing fabrication requirements, the following shall become standard inspection policy:

1) Fillet welds on backing bars for field welded joints shall be 2" long on 8" centers. Fillet welds on backing bars for groove welds made in a fabrication shop, where the backing is to remain in place, shall be full length.

2) Intersecting field welds shall be drilled out with a 1" or 2" hole depending on the size of weld at the intersection. No drilling will be required on intersecting shop welds, nor will drilling be required when a field weld intersects an existing shop weld.

3) Exterior casing welds need not be ground flush. However, weld profile must still meet the requirements of AASHTO/AWS D1.5 (Bridge Welding Code), Section 3.6.

This standard inspection policy is to be effective immediately and enforced on all current and future casing projects.

Richard J. Spring

RICHARD J. SPRING
Senior Materials & Research Engineer

RJS/mdb

Welding of Column Casings

Column casing welds shall comply with the *Steel Structures*[1] and the *Welding*[2] sections of the Standard Specifications (SS). The requirements of the American Welding Society (AWS) D1.5 *Bridge Welding*, **do** apply to the welding of column casings. A column casing is **not** considered a *main member* unless it is designated as such in the contract. Therefore, any column casing not designated as a *main member* on the contract plans is **not** subject to Nondestructive Testing (NDT) other than visual inspection, per AWS D1.5 Section 6.7.1. The base metal testing and preparation, weld quality, welder, welding operator, Welding Procedure Specification qualifications and material requirements established in AWS D1.5 **do** apply for shop and field welding.

[1] SS 2010, Section 55 or SS 2006, Section 55

[2] SS 2010, Section 11 or Special Provisions 2006, Section 8-3

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 180-9 WELDING**
Approved: Original Signed by Dolores Valls **Dolores Valls, Deputy Division Chief** **Offices of Structure Construction**	**Date: October 15, 2002** **Expires: July 1, 2003** **Supersedes: BCM 180-9 Dated February 20, 1998**

Subject: Review of Contractor's Quality Control Plan (QCP) for Field Welding

The following link should be used for assistance in reviewing a QCP. Contact the Structure Materials Representative appropriate to your EA or District and they will specify who should receive the plan.
http://www.dot.ca.gov/hq/esc/Translab/smforms/StructuralMaterialsRepresentatives.pdf

- Form METS QCP-1; "Contractor's QCP Submittal for Field Welding"

This checklist details necessary requirements of the contractor's QCP submittal and should be made available to the contractor at the pre-construction meeting. The form is intended to facilitate the contractor preparation of their QCP and to ensure a complete submittal plan per the contract requirements. The Structure Representative shall review the Contractor's QCP for completeness prior to sending to METS.

- Form METS QCP-5; "METS Letter of Transmittal"

This form will accompany the Contractor's QCP package when returned to the Engineer. The METS Reviewer will evaluate the Contractor's QCP by checking the elements listed on this form.

Attachment

c: BCR&P Manual Holders
Consultant Firms
P. Stolarski, METS
R. Pieplow, HQ Const.

METS QCP-1 - CONTRACTOR'S QCP SUBMITTAL FOR FIELD WELDING

To: _____, Resident Engineer Date of Submittal:

From: _____, Contractor _____,Name of QCM

Contract No.: _____ Welding Firm:

Material to be Welded: ____Piles, ____Reinf. Steel, ____Col. Casings, ____Structural Steel (New & Existing)

NDT Required : ____RT, ____UT,____MT, ____None Name of NDT Firm:_____

Specifications : _____AWS D1.1 ( ), _____D1.4 ( ), _____D1.5 ( ) (Indicate Yr. Edition)

Our Quality Control Plan for field welding to be performed on the subject contract is submitted for your review and approval. The items checked below are submitted herewith.

QCP ITEMS TO BE SUBMITTED AS A MINIMUM	SUBMITTED	**N /A**	For R.E.Use
1. Organization Chart showing all QC Personnel & their duties			
2. Name & Qualifications of QCM			
3. Names of QC Inspectors - Provide copy of CWI Certification for each. No. of submittals _____			
4. Names & Qualifications of Asst. QC Inspectors – Provide Assoc. CWI Cert. or resume. No. of submittals_____			
5. Procedures Manual of NDT Firm to be used. List names of certified personnel, NDT equipment, test procedures, calibration methods, typical reports, and safety procedures.			
6. Certifications for Level II NDT Technicians No. of submittals _____			
7. Methods and frequencies of NDT Inspections			
8. Method of tracking and identifying weld joints & welders			
9. Prequalified WPS (PQR not required) No. of submittals____			
10. WPS' requiring PQR tests. No. of submittals & tests_____			
11. Electrode & Shielding Gas Certs. For each weld process and base metal combination No. of submittals_____			
12. Welder Certifications for each process & position that each will perform. No. of submittals_____			
14. Visual Inspection Procedures (frequencies, gages & tools)			
15. Daily Production & Inspection Log of Welds by QC Inspector			
16. Action Plan for reporting non-conforming welds			
17. Standard Weld Repair Procedures			
18. Certificate of Compliance Form to be used			
19. One copy each of applicable AWS Welding Codes No. furnished _____			
20. Other:			

QCM Signature: _____ _____

(Printed Name)

METS QCP-5 - METS LETTER OF TRANSMITTAL
REVIEW OF CONTRACTOR'S QUALITY CONTROL PLAN FOR FIELD WELDING

To : _____, Res. Engineer Date of this Transmittal :_____

From : _____ ,Section Chief Contract No. :_____

The Contractor's Quality Control Plan submittal has been reviewed.

_____ QCP substantially complies with specification requirements and approval is recommended.

_____ QCP needs to be resubmitted and unacceptable (reject) items corrected as per comments.

General Contractor :_____ Contractor's QCM:_____

Welding Firm :_____ NDT Firm :_____

Materials to be Welded : ____Struct. Steel ____Misc. Steel ____Rebar ____Col. Casings ____H-Pile ____Pipe Pile

Date of Receipt (R.E.) : _____ Date of Receipt (METS) :_____

QCP DOCUMENTS REVIEWED	# OF SUBMITTALS	ACCEPT	REJECT	COMMENTS
1. Organizational Chart - QC Personnel				
2. QC Manager (QCM)				
3. QC Inspectors (Certs. and eye exam)				
4. Asst. QC Inspectors (Certs.)				
5. NDT Firm - Procedure Manual				
6. Weld and Welder ID and Tracking System				
7. WPS/PQR Submittals				
8. Electrode Certs.				
9. Welder Qualifications				
10. Visual Inspection Procedures				
11, Daily Inspection & Production Log (Format)				
12. Non-conformance Report & Action Plan				
13. Standard Weld Repair Procedures				
14. Welding Cert. of Compliance (Format)				
15. Copies of applicable AWS Codes				

NAME OF METS REVIEWER :_____ DATE COMPLETED _____