State of California
Department of Transportation
Division of Engineering Services

Temporary Structures Manual

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Acknowledgements

The 2025 Temporary Structures Manual is the result of Structure Construction and industry partners seeking uniformity and consistency in the administration of projects with various temporary structures. In recent years, the maintenance, rehabilitation, and difficult or limited work access for bridges and other structures has increased the need for various temporary structures such as engineered scaffolding, trestles, temporary highway bridges, guying systems, and bridge supports. The formal Temporary Structures Manual seeks to collect valuable experience and knowledge in these temporary structures and provide that information in a clear, concise edition to field staff and industry partners.

The effort to draft a new Temporary Structures Manual began in earnest in 2022 by the Temporary Structures Technical Team in Structure Construction Office A under the leadership of team sponsor, Bryan Bet, P.E. Principal authors Jim Nicholls, P.E. and Richard Yates, P.E. worked with contributors Lisa Chang-Bridges, P.E., Michael Charak, P.E., Travis Christie, P.E., Kari Forbes, S.E., Alejandro Gomez, P.E., Ramandeep Guraya, P.E., David Rivas, P.E., Høgni Setberg, P.E., Sara Soleimani, P.E., Nathan Quiroz, P.E., and Justin Wood, P.E., to develop the topics, create, and edit the manual. The manual development team is grateful to all the Temporary Structures Technical Team members and industry partners who took the time to provide insightful comments and recommendations. The team would also like to thank Office F for valuable comments from David Tenorio, P.E., Jim Cook, P.E., Alex Lara, P.E., Cliff Law, P.E., David Quintana, P.E., Eric Urmeneta, P.E., Jason Wilcox, P.E., and Jeremy Light, P.E.

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Signed,

JOHN LAMMERS
Deputy Division Chief
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Chapter 1: Introduction

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1-1 Purpose and Scope

The *Temporary Structures Manual* has been issued by the Department of Transportation's Division of Engineering Services (DES), Structure Construction (SC). The intended purpose is to provide an educational resource and administrative support to the SC staff who are in responsible charge of temporary structures on State highway projects. Proper use of the *Temporary Structures Manual* requires a thorough understanding of the principles of civil engineering design and construction, as well as familiarity with the relevant specifications. The Caltrans *Falsework Manual* is referenced several times and is an indispensable resource when reviewing temporary structures with similar design requirements to falsework.

The scope of this manual is to provide a resource for temporary structures, not addressed in either the *Falsework Manual* or the *Trenching and Shoring Manual*, and is not intended to cover all possible scenarios. Engineering judgment will be required to determine the applicability of this manual to a specific temporary structure.

1-2 Statement of Structure Construction Policy

SC policy is to review and authorize temporary structure submittals that potentially create hazardous conditions. This is accomplished by a thorough review and authorization process of the temporary structure shop drawings and verifying that the details and requirements of the authorized shop drawings are properly implemented in the field.

1-3 Contract Specifications and References

Some of the key requirements for the review and authorization of temporary structures can be found in the following *Contract Specifications* and references:

1. Caltrans *Contract Specifications*:
    a. Section 5, *Contract Components*:
        i. Section 5-1.02, *Control of Work – Contract Components*: Item 1.6, *Supplemental Project Information*, includes permits and agreements negotiated by the State that are part of the contract and can include work and restrictions that can have a major impact on temporary structures.
    b. Section 7, *Legal Relations and Responsibility to the Public*:
        i. Notice of responsibility to the public including a section on Cal/OSHA requirements.
        ii. Requirement to submit shop drawings when temporary facilities could be hazardous to the public.

   c. Section 12, *Temporary Traffic Control*:
      i. Temporary access routes for pedestrians must be maintained, and the requirements are found in this section.
      ii. Clearance requirements for traffic control devices.
      iii. Pedestrian route requirements.
   d. Section 14, *Environmental Stewardship*:
      i. Since the majority of bridges are either habitat for biological resources or are adjacent to such habitat, environmental stewardship is a significant consideration in the design and installation of temporary facilities.
   e. Section 16, *Temporary Facilities*:
      i. The requirements for temporary facilities used to move pedestrians through the work site can be found here.
   f. Section 48, *Temporary Structures*:
      i. The requirements for engineering calculations, shop drawings, and review.
   g. Section 60, *Existing Structures*:
      i. This section addresses bridge removal, including protective covers.
2. Project special provisions:
   a. The project special provisions will contain contract-specific requirements and restrictions above and beyond what is in the *Standard Specifications* and will frequently include requirements outlined in the project permits and agreements.
3. *Information Handout*:
   a. The *Information Handout* contains the project-specific permits and agreements including railroad agreements, environmental permits, asbestos report, and lead report when applicable.

1-4 Contractual Relationships

The Contractor is responsible for designing and implementing the temporary structure shop drawings. The Structure Representative is responsible for reviewing and authorizing the shop drawings and verifying that the Contractor follows the authorized plan. The Contractor’s engineer that signed and sealed the temporary structure shop drawings is the engineer in responsible charge of the work.

1-5 Cal/OSHA References

There are many specific Cal/OSHA regulations covering temporary structures and those references will not be covered in this section specifically. Rather, this section highlights some general areas of importance regulated by Cal/OSHA in relation to temporary structures. The specific requirements for each type of structure will be covered in the associated chapter in this manual. The primary requirements associated with temporary structures are:

1. Structure collapse
2. Falls from elevated work areas
3. Minimum loading requirements
4. Public safety
5. Heavy construction equipment and traffic.

1-6 Definitions

As-Builts – Historical record of a bridge's design and modifications implemented during construction or maintenance.

Authorized reviewer – The structure representative or their delegated assistant in responsible charge of reviewing the temporary structure submittal.

Cal/OSHA – **California Department of Industrial Relations, Division of Occupational Safety and Health (DOSH)** – Cal/OSHA protects and improves the health and safety of working men and women in California.

Contract documents – The various items described in *Standard Specifications* Section 5-1.02, *Control of Work – Contract Components*, including the special provisions, project plans, standard specifications, standard plans, change orders, information handout, permits, licenses, agreements, and certifications that encompass the definition and scope of work as agreed to by Caltrans and the contractor.

Construction Safety Orders (CSOs) – California Code of Regulations, Title 8, Chapter 4, Subchapter 4 provides safety regulations for the construction industry.

DES – Caltrans Division of Engineering Services. This is the division of Caltrans responsible for the design and construction engineering/contract administration of bridges and structures within Caltrans right-of-way. This includes oversight of bridges constructed by other agencies within Caltrans right-of-way or by other agreement outside Caltrans right-of-way.

Protective cover – Shielding installed between bridge removal work and any resource, utility, or public area to be safeguarded or preserved.

SC – Structure Construction. This is the subdivision within Caltrans DES responsible for the administration of bridge (and other structures) construction contracts within Caltrans right-of-way or oversight thereof.

Temporary bracing – Short-term structural support, primarily to resist lateral forces. Bracing should be designed and constructed to resist actual horizontal forces and the minimum contractually specified horizontal forces.

Temporary support – Short-term structural support to an existing structure or structural element during construction activities.

Traffic opening – Provisions for the passage of traffic through temporary works, including minimum clearances, impact-resistant elements, and lighting.

Chapter 2: Review and Authorization of Temporary Structure Submittals

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2-1 Introduction

This chapter covers Structure Construction (SC) practice with respect to the temporary structure submittal review process. Subsequent chapters cover specific review guidelines, procedures, and explanations where necessary to ensure uniform and impartial contract administration. As noted in Chapter 1, *Introduction*, review and authorization of the Contractor's temporary structure submittal is delegated to SC's Structure Representative in responsible charge of structure work at the project site. While the actual review of the temporary structure submittal may be performed by a qualified member (authorized reviewer) of the project team, the Structure Representative is expected to give their personal attention to the review while it is in progress and provide concurrence before the submittal is authorized.

The contract requirement for submission of temporary structure submittals should be discussed with the Contractor at the preconstruction conference, with emphasis on the need for a complete submittal before the review period begins. If temporary structures are subcontracted, strongly encourage the prime Contractor to have their subcontractor attend the preconstruction conference. The Contractor should be reminded that erection of temporary structures must not begin until the shop drawing is authorized.

2-2 Design Calculations and Shop Drawings

Submittal of shop drawings for review and authorization is required by the contract documents when temporary structures create a hazardous condition to the public. The *Contract Specifications* requires that the design calculations demonstrate design adequacy of the load-supporting members. In the specification context, the term "load-supporting members" will be construed as meaning the design-controlling members. Temporary structures are designed in accordance with *Contract Specifications* Section 48, *Temporary Structures*. The Caltrans *Falsework Manual* is an essential tool in the administration of Section 48, although loading requirements may differ when compared to structures that are not falsework. Temporary structures that are not addressed in Section 48 should be designed using the best industry practice.

The design calculations furnished by the Contractor are for information only; they are not for review and authorization. Any required design or construction details which may be shown in the form of sketches on calculation sheets must be included in the temporary structure shop drawings; otherwise, the plan is incomplete. Temporary structure submittals are not to be authorized in any case where it is necessary to refer to calculation sheets for information needed to complete the independent design review, or where information shown only on the calculation sheets will be needed for construction. In most cases, it is unnecessary to refer to the Contractor's calculations during the design review. However, in the event a load-supporting member is overstressed or is otherwise determined to be inadequate, reference to the calculations may reveal the reason for the design deficiency.

2-3 Temporary Structure Submittal Review

2-3.01 Initial Review

Immediately upon receipt of a temporary structure submittal, the authorized reviewer will perform an initial review of the documents received. The purpose of the initial review is to ascertain whether the plan and all required supporting data are included in the submittal. Determining whether the submittal is complete involves a certain degree of subjectivity, and the Engineer will be expected to exercise judgment when making this determination. The basic requisite is that the plan contains enough information to enable the Engineer to verify that the design meets the contract requirements. Although the initial review is not a contractual requirement, it is the practice of SC to identify incomplete or deficient submittals early in the review process.

The temporary structure submittal should include the following:

1. Details of associated temporary structure activities, including controlling dimensions.
2. Methods and sequence of construction, including staging and equipment locations.
3. Locations where work is performed over traffic, utilities, or railroad property
4. Protection of people, property, utilities, and improvements.
5. Methods for preventing material, equipment, and debris from falling onto traffic, railroad property, or other protected area.
6. Methods for removing temporary structures.

The initial review is to be completed within two working days following the receipt of the temporary structure submittal. The purpose of this is to ensure timely notification to the Contractor in the event the drawings are not complete. Since the only purpose of the initial review is to discover omissions that would prevent completion of a subsequent design check, neither calculations nor an evaluation of design details is required; thus, completion within two working days is reasonable.

2-3.02 Review

The importance of having a complete plan and thorough review cannot be overstated when it comes to temporary structure submittals, as the effort invested in preparation and review of the temporary structure submittal pays dividends when field work commences.

2-3.02A Procedure when Railroad Company is not Involved

Except for work that is adjacent to or over a railroad, the temporary structure submittal may be authorized when the Structure Representative is satisfied that the submittal meets all contract requirements. Authorization should follow the procedure discussed in Section 2-3.03, *Engineering Analysis*. Each sheet of the shop drawings or the cover sheet of work plans must be signed by the Structure Representative or authorized reviewer. One set of the authorized temporary structure submittal will be returned to the Contractor, with an authorization cover letter signed by the Structure Representative.

2-3.02B Procedure when Railroad Company Approval is Required

In order to expedite the review process of the temporary structure submittal by railroad companies, it is advisable that the drawings submitted by the Contractor adhere to the requirements of the guidelines produced by the associated railroad. The two main railroad guidelines are listed below:

- Union Pacific Railroad – BNSF Railway: *Guidelines for Railroad Grade Separation Projects*
- American Railway Engineering and Maintenance-of-Way Association (AREMA): *Manual for Railway Engineering (MRE).*

Other railroads not mentioned above may have their own specific guidelines and requirements.

The special provisions will also list any clearance requirements that need to be maintained. These measurements are taken from the centerline of the railroad tracks. If there are no clearances included in your contract documents, refer to guidelines produced by the associated railroad.

Where there is a conflict between the *Contract Specifications* and the guidelines issued by the railroad, the *Contract Specifications* must prevail.

2-3.02C Railroad Requirements

Some common requirements are often overlooked and have resulted in submittals being returned by the railroad. The temporary structure submittal must state that all temporary structure construction will comply with the latest railroad guidelines and AREMA. The temporary structure submittal must note how the Contractor will gain access to the site, staging for all materials, equipment locations when working, and staging areas for equipment. Track protection details are outlined in the aforementioned guidelines, and details must be included on the plans.

The temporary structure submittal must note if there are any existing drainage facilities, including drainage ditches, or access roads being affected by the Contractor's operations. If there are no existing drainage facilities or access roads, the drawings must note this fact. Railroad personnel who are unfamiliar with the site often review the temporary structure submittal.

The above railroad requirements must be discussed at the preconstruction conference with the Contractor. Remind the Contractor that approval of temporary structure submittals over and/or adjacent to railroad tracks will be contingent upon the railroad approving the plans.

2-3.02D Distribution of Temporary Structure Submittals

The Structure Representative will review the temporary structure submittal and if necessary, return it to the Contractor for correction. After the Structure Representative is satisfied that the submittal meets the specification requirements, send the following items to SC Falsework Engineer in SC Headquarters:

1. The Contractor's temporary structure shop drawings or work plan.
2. The Contractor's calculations.
3. The Structure Representative's calculations.
4. Manufacturer's data relative to all manufactured devices.

Note: The Structure Representative must not stamp the temporary structure submittal "Authorized" until SC Falsework Engineer has notified them that the railroad has reviewed and authorized the plans.

2-3.02E Railroad Review and Authorization

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

Please note that all correspondence with the railroad regarding the status of submittals under their review must be directed to the SC Falsework Engineer. At the railroad's request, under no circumstances should you contact the railroad directly.

When the railroad review is complete and determines the plans to be acceptable, the railroad notifies the SC Falsework Engineer, who will advise the Structure Representative to proceed with authorization of the temporary structure submittal. The Structure Representative will then stamp the plans "authorized" with the date of authorization and return them to the Contractor along with the temporary structures analysis report; note that this is an engineering analysis report. Assuming proper notification has been made to the railroad that their horizontal and vertical clearances will be impaired and that a flagger is required, the Contractor may begin work. Refer to the Railroad Agreement in the contract *Information Handout* for detailed requirements. Note that the Contractor **must not begin** any operations within the railroad right-of-way until the authorized plans have been issued to the Contractor.

2-3.03 Engineering Analysis

The temporary structure submittal is authorized pursuant to *Contract Specifications* Section 5-1.23, *Control of Work – Submittals,* which includes requirements for the review duration. The review durations provided are for the majority of temporary structure submittals; check the special provisions if there is railroad involvement or other factors (such as a prefabricated modular truss panel) that may necessitate a longer review period.

SC's practice is to perform an independent engineering analysis on temporary structure submittals that are required to be sealed and signed by a civil engineer registered in the State of California. The independent review can be a simple review or a complex analysis with assistance from the Bridge Design Engineer, Bridge Construction Engineer, or the SC Falsework Engineer.

Upon completing the engineering analysis of the temporary structure submittal, the Engineer is to present the findings in an engineering analysis report. The report is to be sealed and signed in accordance with the Professionals Engineers Act (Business and Professions Code), § 6735. Refer to the *Falsework Manual* Section 1-10, *State Statutes,* for further information.

The engineering analysis report is an engineering document and therefore must be sealed and signed by a civil engineer registered in the State of California in responsible charge of the independent engineering analysis.

This report is to be completed for authorized and rejected temporary structure submittals. The report is to contain a brief chronological record of the pertinent dates related to the submission, review, rejection (if applicable) and authorization of the plan, including the number of review days. The Structure Representative is to transmit the report to the Contractor through the project's normal transmittal process. An example of the temporary structure analysis report is provided in Section 2-3.04, *Sample Engineering Analysis Report*.

When the shop drawings or work plan cannot be authorized, complete the engineering analysis report and list the reason(s) that the shop drawings or work plan are rejected. Elaboration is unnecessary and corrective measures should not be suggested. Prior to sending the report to the Contractor, contact the temporary structure engineer of record by phone or in person to discuss the reason(s) for rejecting the submittal. Document this discussion in the chronological record and the daily report.

When the shop drawings or work plan is authorized, complete an engineering analysis report. The report must include the following paragraphs:

1. "The <*insert type of submittal, trestle, scaffold, shop drawings, work plan, etc*> for <*identify specific location*> of the <*bridge name and number*> is found acceptable based on an independent engineering analysis and is authorized to the extent provided in the *Contract Specifications* Section 5-1.23, *Control of Work – Submittals.*"
2. "Your attention is directed to your responsibilities pursuant to *Contract Specifications* Sections 5-1.23, *Control of Work – Submittals*, 7-1.04, *Legal Relations and Responsibility to the Public – Public Safety*, and Section 48, *Temporary Structures,* and to the applicable requirements of the *Construction Safety Orders*."
3. "You are reminded that <*insert type of submittal, trestle, scaffold, shop drawings, work plan, etc.*> construction must conform to the authorized submittal, that the materials used must be of the quality necessary to sustain the stresses required by the design, and that workmanship must be of such quality that the <*insert type of submittal, trestle, scaffold, shop drawings, work plan, etc.*> will support the loads imposed without excessive settlement or joint take-up beyond that shown on the authorized <*insert type of submittal, trestle, scaffold, shop drawings, work plan, etc.*>."

A sample engineering analysis report is provided below and can be used as a template.

2-3.04 Sample Engineering Analysis Report

Engineering Analysis Report

<Insert Date>

Project Information

Contract Number
Dist-Co-Rte-PM
Bridge Name
Bridge Number

Type of structure reviewed: *<Insert type of temporary structure>*

Chronology

Plans were received: *<date>*
Plans rejected: *<date>*
Revision No. 1 received: *<date>*
Revision No. 1 rejected: *<date>*
Revision No. n received: *<date>*
Revision No. n rejected: *<date>*
Plans authorized: *<date>*
Elapsed review time: *<calendar days>*

Introduction

This report presents the results of an independent engineering analysis of the *<insert type of review completed>* for *<bridge name, bridge number, and specific location >.*

Discussion

<u>*Rejection*</u> – *This portion of the report describes specific deficiencies found with the temporary structure submittal that would be cause for rejection.*

For clarity, redline clouds may be made on the temporary structure submittal and then described here.

<u>*Authorization*</u> – No exceptions were found.

Conclusion

Rejection:

The <*insert type of submittal, trestle, scaffold, shop drawings, work plan, etc.*> for <*identify specific location*> of the <*bridge name and number*>, is rejected based on an independent engineering analysis. The deficiencies are listed above.

Authorization (the paragraphs below must be included):

“The <*insert type of submittal, trestle, scaffold, shop drawings, work plan, etc*> for <*identify specific location*> of the <*bridge name and number*> is found acceptable based on an independent engineering analysis and is authorized to the extent provided in *Standard Specifications* Section 5-1.23, *Control of Work – Submittals*.”

“Your attention is directed to your responsibilities pursuant to *Contract Specifications* Sections 5-1.23, *Control of Work – Submittals*, 7-1.04, *Legal Relations and Responsibility to the Public – Public Safety*, and Section 48, *Temporary Structures,* and to the applicable requirements of the *Construction Safety Orders*.”

“You are reminded that <*insert type of submittal, trestle, scaffold, shop drawings, work plan, etc.*> construction must conform to the authorized submittal, that the materials used must be of the quality necessary to sustain the stresses required by the design, and that workmanship must be of such quality that the <*insert type of submittal, trestle, scaffold, shop drawings, work plan, etc.*> will support the loads imposed without excessive settlement or joint take-up beyond that shown on the authorized <*insert type of submittal, trestle, scaffold, shop drawings, work plan, etc.*>.”

If you have any questions regarding this report, please contact <*insert Structure Representative or authorized reviewer’s name*> at (XXX) XXX-XXXX.

<Signature of reviewer>
Loren N. Bridge, P.E.
Structure Representative
Structure Construction

2-4 Safety and Cal/OSHA Requirements

All construction safety standards apply to temporary structure work.

Cal/OSHA regulations contain many safety requirements associated with temporary structures and it is not the intent of this section to cover all the Cal/OSHA regulations, but rather to remind the Structure Representative of these requirements.

Although the Structure Representative has neither the authority nor the duty to enforce this Article, as a matter of policy, verification that the Contractor has a valid permit will be done before the submittal is authorized in any case where a permit is required. The date of verification should be noted in the project diary.

Pertinent Cal/OSHA requirements should be discussed with the Contractor at the pre-construction conference.

The construction of temporary structures requires attention to fall hazards and possible hazards of working over water. All elevated work is subject to gravity and the stored energy in an elevated mass. Personnel fall hazard protection and falling debris control require planning and are addressed in the authorized temporary structure work plans and supporting documents. The site is frequently changing, noisy, sometimes dusty, and often congested with equipment working at a fast pace. Competent supervision is essential to safely accomplish the construction goals.

2-5 Design Revisions to Authorized Plans

Design revisions to the authorized plans may occur for many reasons including the following: the Contractor decides to change a particular means or method, something is identified during construction that was not known before, an unplanned event occurs, or the temporary structure deviates from what was authorized.

If an unplanned event occurs or the temporary structure deviates from what was authorized, the Contractor must immediately stop work and submit procedures to correct or remedy this occurrence.

Administratively, and as defined in *Contract Specifications* Section 5-1.23B, *Control of Work – Submittals – Action Submittals*, any revision to an authorized temporary structure submittal will be viewed as a new submittal, and as such will be reviewed pursuant to the applicable specification requirements.

The Contractor must show the revision number on the revised temporary structures submittal, uniquely number each revised detail, and describe and date the revisions in a legend. The revision is to be identified with an inverted triangle or revision cloud. A complete submittal must be provided with each revision to the temporary structures submittal. Refer to *Contract Specifications* Section 5-1.23B(2), *Control of Work – Submittals – Action Submittals – Shop Drawings*.

Chapter 3: Design Considerations

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3-1 Introduction

Loading for temporary structures varies depending on the type of structure and the requirements specified in the contract specifications. The *Contract Specifications* can require minimum horizontal and vertical design loads; however, in some cases, the design loads are determined by the design engineer. This chapter will address typical loads and load combinations that may be encountered in the submittal review of temporary structures, but not all loading conditions will be discussed. Each unique structure will require the loading be applied according to the contract specifications, if available, or loading in compliance with design criteria or codes appropriate for the facility involved.

3-2 Contractual Requirements

Contract Specifications Section 7-1.04, *Legal Relations and Responsibility to the Public - Public Safety,* states:

> *Temporary facilities that could be a hazard to public safety if improperly designed must comply with design requirements described in the Contract for those facilities or, if none are described, with standard design criteria or codes appropriate for the facility involved. Submit shop drawings and design calculations for the temporary facilities and show the standard design criteria or codes used. Shop drawings and supplemental calculations must be sealed and signed by an engineer who is registered as a civil engineer in the State.*

Loading for most temporary structures is addressed in the *Contract Specifications* Section 48, *Temporary Structures,* or as shown on the contract plans. When loading is not described, best industry practice should apply.

3-3 Cal/OSHA Requirements

Requirements from Cal/OSHA's Construction Safety Orders vary depending on the type of structure being considered. Most temporary structures require the design to be by a qualified person or licensed engineer. Some structures, such as scaffolding, are required to conform to listed loading and load combinations. Note that the Construction Safety Orders are found in the California Code of Regulations, Title 8, Division 1, Chapter 4, Subchapter 4.

3-4 Structure Categories

Many structures are similar in design but have unique loading and design requirements associated with their use. Some typical temporary structures are listed below, along with the associated design considerations.

Falsework is typically associated with the construction of cast-in-place concrete structures, particularly bridge structures. In this type of construction, falsework provides a stable platform upon which the forms may be built and furnishes support for the bridge superstructure until the members being constructed have attained sufficient strength to support themselves. Section 48-2, *Temporary Structures – Falsework*, of the *Contract Specifications* contains specific loading and design criteria for falsework. The *Falsework Manual* is another source for the design criteria for falsework.

Temporary supports are similar to falsework and are typically used to provide support for permanent structures during retrofit, reconstruction, erection, and removal activities. Many accelerated bridge construction (ABC) projects require some form of temporary support during erection. Section 48-3, *Temporary Structures – Temporary Supports*, of the *Contract Specifications* contains specific loading and design criteria for temporary supports which are discussed in detail in Chapter 9, *Temporary Supports and Jacking,* of this manual.

Trestles and work platforms are used to support equipment loads and other construction activities. The design criteria and loading for these structures are typically provided by the temporary structure designer. These types of structures are discussed in detail in Chapter 4, *Temporary Access Trestles,* of this manual.

Scaffolding, as defined by Cal/OSHA, is any temporary elevated platform and its necessary vertical, diagonal, and horizontal members used to support workers and materials. Chapter 7, *Bridge Scaffolding*, discusses the design of bridge scaffolds in more detail.

Protective covers are associated with bridge removal operations to collect debris. Chapter 5, *Design Considerations*, of the *Bridge Removal Manual* addresses protective covers. Loading for protective covers is typically provided by the design engineer. Protective covers are also required when pedestrian routes pass beneath work areas such as falsework.

Some temporary structures are used for more than one purpose and should be designed to the most restrictive requirements. For example, scaffolding is often used as a protective cover in addition to providing a platform for workers.

3-5 Dead Loads

The dead load is typically the self-weight of the supported structure in place at any time in the construction sequence. The construction dead loads include the weight of any forms, shoring, or other temporary structures in place during the construction sequence. The total dead load for temporary structures is typically the summation of the two loads described above.

Similar to values used in falsework analysis, the weight of the concrete, forms, and reinforcing steel is:

- 160 pounds per cubic foot (pcf) for normal concrete.
- 130 pcf for lightweight concrete.

As a proportion of concrete weight, the weight of forms and rebar for typical concrete bridges may be estimated as:

- 15 pcf

The densities of other common building materials can be found in Table 17-12, *Densities of Common Materials*, of the *AISC Steel Construction Manual* or in other references.

Other items considered as dead load in design of temporary structures, in addition to the weight of the temporary structure or the in place supported structure, can be demolition debris, stored materials, and construction equipment.

3-6 Live Loads

Live loads are determined by the design engineer unless specified in the contract documents. Minimum live loads are determined by the type of temporary structure and the intended use. The minimum live load for falsework is 20 psf and this value is also appropriate for other temporary structures. Scaffolding requires higher minimum live loads and is discussed elsewhere in this manual. Live loads due to equipment such as cranes are determined by adding the actual equipment weight plus the maximum loads it will support. Live loads from equipment are moving loads that must be analyzed in the position that produces the maximum stresses to the supporting structure.

3-7 Environmental Loads

3-7.01 Wind Loads

There are many methods available to calculate wind loads. The method used for calculating the wind load must be appropriate for the type and shape of structure being analyzed. This section will discuss a few of the more common methods for calculating wind loads. If the method for calculating the wind loads is not specified in the contract documents, then it is the responsibility of the design engineer to select the appropriate method. The design method used for calculating wind loads, along with any assumptions, should be noted on the shop drawings.

The simplest method for determining the wind pressure on a temporary structure is the load tables found in the *Contract Specifications* Section 48-2.02B(2), *Temporary Structures –Falsework – Materials – Design Criteria – Loads*. The wind pressure values in this section are applicable to falsework but can also be applied to other temporary structures that are similar to falsework. The wind pressures are applied to the projected area of the structure.

Another commonly used method for determining wind loads is found in the American Society of Civil Engineers (ASCE) *Minimum Design Loads and Associated Criteria for Building and Other Structures.* Discussions and examples used in this manual will assume the most recent version is used, which is ASCE 7-16. The methods found in ASCE 7-16 are much more complicated than the pressure tables found in the *Contract Specifications*. The procedures in ASCE 7-16 are primarily developed for building design, and care should be taken that the proper procedure is used for temporary structures. The wind pressures are applied to the structure as directed in the ASCE 7-16.

The procedures found in the American Association of State Highway and Transportation Officials (AASHTO) *Guide Design Specifications for Bridge Temporary Works* (GSBTW) are similar to the procedures in ASCE 7-16, but are based on the procedures found in the AASHTO *Bridge Design Specifications* (BDS). The BDS methods are currently based on ASCE 7-10, but pending revisions will adopt the procedures in the current ASCE 7. The GSBTW has simplified the determination of the variable used in the wind pressure calculations for temporary structures. The wind pressures are applied to the projected area of the structure.

See Appendix A Example 1*, Wind Loads,* for sample calculations and a comparison of the wind pressure methods described above.

3-7.02 Seismic Loads

Seismic loads are typically not applied to temporary structures due to the short duration the temporary structure is in service. Some minimal lateral loads are increased, such as for temporary supports, to account for the statistical probability of a seismic event based on anticipated service life.

When seismic loads are applied to temporary structures, the loads are typically based upon a 50-year seismic event (2 percent probability event).

Procedures for calculating seismic loads can be found in ASCE 7. The AASHTO GSBTW provides modifications to the ASCE 7 procedure that are applicable to temporary structures.

3-7.03 Stream Flow

When temporary structures are placed in flowing water, the water pressure applied to the temporary structure is:

$$\mathbf{P_W = Kv^2} \tag{3-7-1}$$

$\mathbf{P_w}$ = pressure (psf)

v = water velocity (ft/s)

K = 1.375 for square faces

0.67 for circular piers

0.50 for angular faces

The formula above is based on Section 2.3.5.5, *Stream Flow*, of the AASHTO GSBTW.

If significant drift buildup is anticipated, the potential increase in loading should be investigated.

3-7.04 Soil Pressure

Soil pressure can be determined using the methods and guidelines found in the Caltrans *Trenching and Shoring Manual*.

3-8 Load Combinations

Temporary structures are typically designed using Allowable Stress Design (ASD); however, in some circumstances the designer may elect to use Load and Resistance Factor Design (LRFD). The load combinations discussed in this section will be based on ASD. For LRFD combinations, refer to AASHTO GSBTW Table 2.3.2.2-1, *Load Combinations and Load Factors*. The typical ASD load combinations considered for the design of temporary structures (except falsework) are as follows:

$\mathrm{DL+LL}$
$\mathrm{DL+0.75LL+0.75WL}$
$\mathrm{DL+WL}$
$\mathrm{0.6DL+WL}$

DL – Dead Load
LL – Live Load
WL – Wind Load

The combinations above represent the typical loading conditions and are not all-inclusive. The effect of one or more loads not being applied should be considered to determine the most unfavorable loading condition.

Assumed minimum horizontal loads, such as the 10 percent minimum load used in temporary support design, are not reduced when combined with dead or live loads.

When calculating wind loads using the methods found in ASCE 7, it should be noted that these wind loads are based on LRFD and should be reduced by 0.60 for ASD. The combination above with the 0.60 factor applied would be as follows:

$$\mathrm{DL+LL}$$
$$\mathrm{DL+0.75LL+0.75(0.6)WL}$$
$$\mathrm{DL+0.6WL}$$
$$\mathrm{0.6DL+0.6WL}$$

When stream flow or horizontal earth pressure is applied, the typical combinations for ASD are as follows:

$$\mathrm{DL+LL+WA+EH}$$
$$\mathrm{DL+0.75LL+0.75(0.6)WL+WA+EH}$$
$$\mathrm{DL+0.6WL+WA+EH}$$
$$\mathrm{0.6DL+0.6WL+WA+EH}$$

WA – Stream Flow
EH – Horizontal Earth Pressure

3-9 Traffic Openings

Whenever an operation will reduce clearances available to public traffic, the *Contract Specifications* Section 7-1.04, *Legal Relations and Responsibility to the Public – Public Safety,* requires the Contractor to notify the Resident Engineer within a specified timeframe before the anticipated start of the operation. Moreover, the *Contract Specifications* Section 12-4.02A(3)(b), *Temporary Traffic Control – Maintaining Traffic – Traffic Control Systems – General – Submittals – Closure Schedules*, requires the Contractor to submit a closure schedule request within a certain timeframe before the anticipated start of any job site activity that reduces horizontal or vertical clearance of traveled ways.

Referring to BCM C-6, *Required Documents to be Submitted During Construction*, the Structure Representative completes Form TR-0019, *Notice of Change in Clearance or Bridge Weight Rating,* or Form TR-0029, *Notice of Change in Clearance or Bridge Weight Rating,* as applicable. The form is submitted to the Resident Engineer as notification of the change. The Resident Engineer notifies the Transportation Permits

Branch. After erection of the temporary structure, the Structure Representative verifies the clearance.

The minimum width and height of each traffic opening to be provided through the temporary structure will be shown on the structure contract plans or in the special provisions.

Chapter 4: Temporary Access Trestles

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4-1 Introduction

Temporary access trestles are essential in bridge construction projects where standard access methods are insufficient due to challenging terrain or environmentally sensitive areas. These structures provide the necessary support and accessibility for heavy equipment in locations such as above-water bodies, rough terrains, or environmentally sensitive areas where conventional access methods might be impractical, damaging, or prohibited according to governing agency permits. Trestles facilitate efficient and safe access to the work site, significantly reducing the reliance on floating equipment like barges or high-line systems that span canyons and valleys.

The basic makeup of a standard trestle is similar to falsework and typically includes driven or drilled piles for foundation support, steel beams for the bent caps and stringers, and wood decking for the platform. Bracing is normally provided by steel wire rope or structural steel members. These elements are designed to withstand various forces, including the weight of heavy equipment, dynamic movements, and environmental loading, such as water flow and wind. Proper design and construction of trestles are essential to ensure stability, safety, and environmental compliance during the project execution.

Contractors typically design temporary access trestles as part of their means and methods for building the permanent structure. It is worth noting that the term “trestle” is not found in the *Standard Specifications*.

Figures 4-1, 4-2, 4-3, 4-4, 4-5, and 4-12 show various examples of temporary access trestles used in bridge construction.

Figure 4-1. Antlers Bridge, I-5, CA

Figure 4-2. Shasta Viaduct Arch Bridge, I-5, CA

Figure 4-3. Dr. Fine Bridge, Smith River, U.S. 101, CA

Figure 4-4. HWY 97 at the Link River, OR

Figure 4-5. Sacramento River Bridge at Butte City, HWY 162, CA

4-2 Contractual Requirements

The *Contract Specifications* Section 7-1.04, *Legal Relations and Responsibility to the Public – Public Safety*, requires that temporary facilities which could pose a public safety hazard if improperly designed must adhere to the design requirements specified in the contract or, if not specified, follow standard design criteria or codes relevant to the facility. Shop drawings and design calculations for these facilities must be submitted to the Engineer for authorization, indicating the design criteria or codes used. All shop drawings and calculations must be sealed and signed by a registered civil engineer in the State.

Temporary access trestles fall under these public safety requirements and will be managed accordingly. The signed and sealed shop drawings and calculations will be considered an action submittal as outlined in *Contract Specifications* Section 5-1.23B, *Control of Work – Submittals – Action Submittals*. The Contractor's engineer who signed and sealed the temporary structure shop drawings is the engineer of record.

Construction projects in or near environmentally sensitive areas, such as bodies of water, wetlands, or steep terrain, often face unique environmental challenges. These areas may host protected plant, fish, and wildlife species and be subject to noise and vibration restrictions. Pile driving in riverbeds can harm fish habitats, and permit restrictions may require the use of bubble curtains or other methods to reduce vibration and sound waves in water or limit in-water work to specific timeframes.

The *Information Handout* may include project-specific permits and agreements that are part of the contract and must be adhered to. Permitting agencies that may influence the Contractor's methods, design, or installation of the trestles include:

1. US Army Corps of Engineers
2. Federal Emergency Management Agency - FEMA
3. Water Quality Control Board - WQCB
4. Department of Fish and Wildlife - DFW
5. California Coastal Commission - CCC
6. US Coast Guard
7. Other permitting agencies.

4-3 Cal/OSHA Requirements

The Construction Safety Orders issued by Cal/OSHA include various provisions which apply to the design and construction of temporary access trestles, including the following:

- Article 24, *Fall Protection:*

  § 1669, *General.*
  Employers must provide fall protection for employees exposed to falls of 7.5 feet or more. This section covers general fall protection requirements, including the use of guardrails, safety nets, and personal fall arrest systems.

  § 1670, *Personal Fall Arrest Systems, Personal Fall Restraint Systems and Positioning Devices*.
  Personal fall protection systems, such as harnesses and lanyards, must be used where fall hazards exist. This section covers using, maintaining, and inspecting personal fall protection systems, including harnesses, lanyards, and anchorage points.

- Article 13, *Work Over or Near Water*, § 1602, *Work Over or Near Water:*
  Requires the following safety measures for employees working with a risk of drowning: Personal Flotation Devices (PFDs), Ring Buoys, and Lifesaving Boats.

- Article 15, *Cranes and Derricks in Construction*, § 1610, *General Requirements.*

4-4 Review and Authorization

Temporary access trestle structures are reviewed and authorized, as required by *Contract Specifications* Section 7-1.04 *Public Safety*, in accordance with BCM C-11, *Shop Drawing Review of Temporary Structures.*

The initial review of the submittal for completeness should be performed within two working days and ensure the following items are included:

1. Legible drawings and calculations.
2. Stamped by a California licensed engineer.
3. Identification of all components, materials, and information for manufactured assemblies.
4. All controlling dimensions are shown on the shop drawings.
5. Design loads, including live loads, horizontal loads, and environmental loads including water and wind loads.
6. Design criteria or design codes used.
7. Welding design and inspection standards.
8. Foundation installation procedures for driven or drilled piles.
9. Site-specific requirements may include traffic openings, navigation openings, flood contingency plans, allowable work windows, utility restrictions, or restrictions due to a levee or railroad.
10. Erection and removal procedures.

An independent analysis by Caltrans must be performed to verify if the design of the temporary access trestle conforms to the contract specifications and permit requirements, and member stresses are within allowable limits. The analysis findings are to be presented to the Contractor in a temporary structure analysis report signed and sealed by the licensed engineer performing the review.

A detailed work plan is required for temporary access trestle submittals that involve the railroad. The work plan must include the following:

1. Crane pick plan for erection and removal of the temporary access trestle when there is a possibility the crane could foul the railroad tracks.
2. Staging areas for equipment and materials.
3. Path of travel for equipment in and out of railroad right-of-way.
4. Critical dimensions, including dimensions to the centerline of the railroad tracks.

4-5 Basic Components of a Temporary Access Trestle

Foundation: Trestles are usually supported by driven or drilled piles, such as pipe piles, H-piles, or wide-flange beams, to handle the substantial loads they bear. These foundations are essential for supporting heavy cranes.

Cap Beams: These lateral members distribute loads from stringers or girders to the supporting piles.

Stringers/Girders: Spanning between bent caps, stringers and girders are typically laterally braced to maintain stability.

Transverse and Longitudinal Bracing: Transverse and longitudinal (lateral) bracing, made from steel wire rope, structural steel members, or steel beams, handles horizontal and longitudinal loads such as equipment live loads, wind, and other forces. This bracing ensures the trestle remains stable under varying conditions.

Decking: Usually constructed from timber or precast concrete, decking supports the equipment loads. Sealing of the deck may be required to prevent debris from falling through gaps.

Figures 4-6 and 4-7 show the typical components of a temporary access trestle.

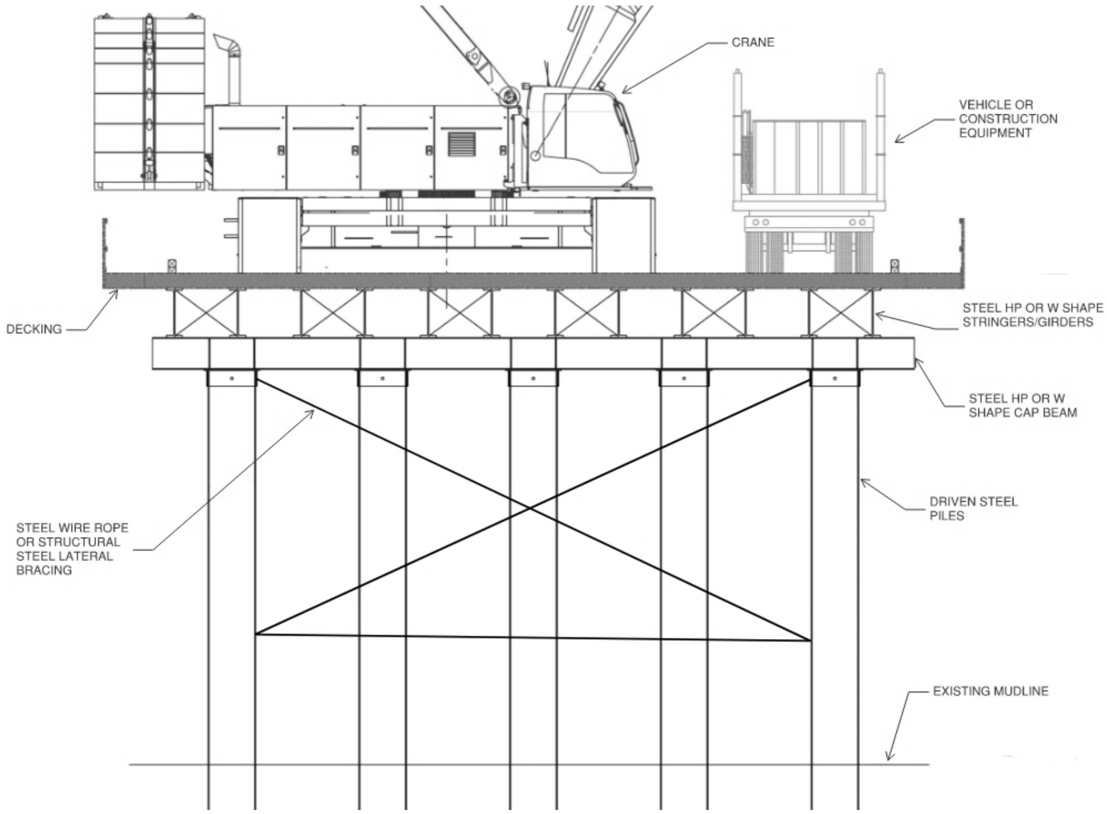


Figure 4-6. Typical Trestle Components, Section View

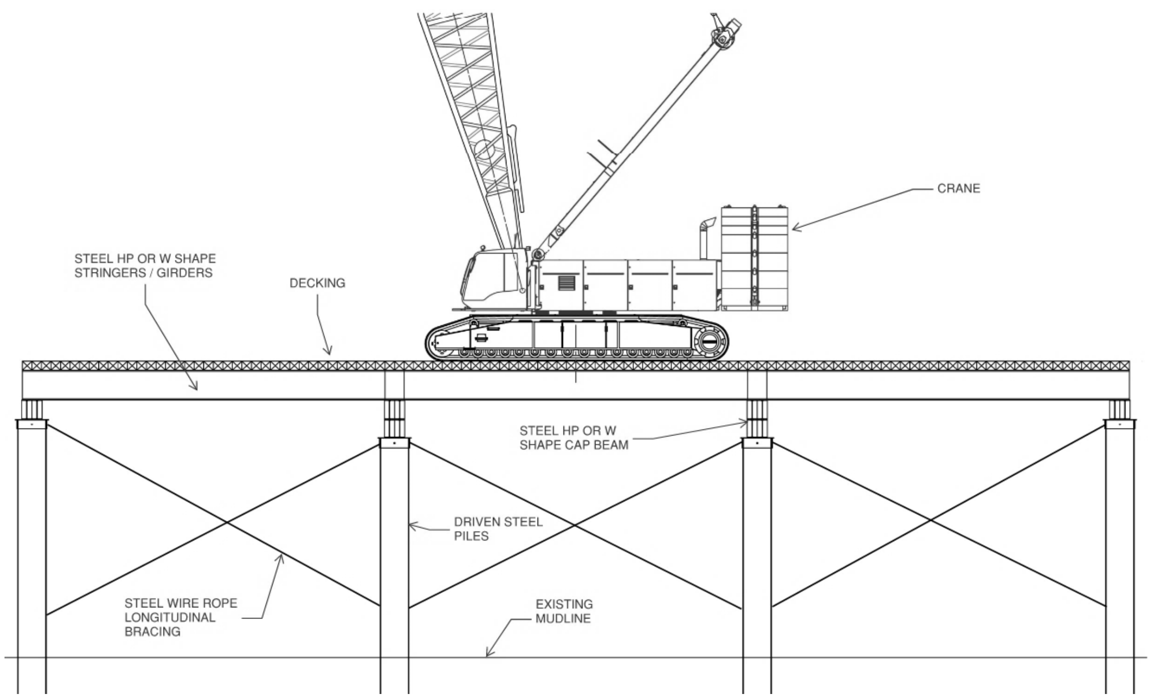


Figure 4-7. Typical Trestle Components, Elevation View

4-6 Loads

Temporary access trestles are generally designed to support the following loads:

1. Dead load: Gravity weight of all the trestle components.
2. Live load: The total weight of the crane plus its maximum lifting load along with other equipment, vehicles and material storage.
3. Live load lateral forces: Arising from the dynamic movements of the crane or other equipment.
4. Water-induced lateral forces: Resulting from water movement, such as waves or currents in a river.
5. Wind lateral forces.
6. Minimum lateral load: Designated by the trestle design engineer.

Access trestles are generally designed to support substantial equipment loads. The design must consider the type of construction equipment, its placement on the trestle, and the sequence of operations to ensure that the supporting members can handle the maximum expected stresses.

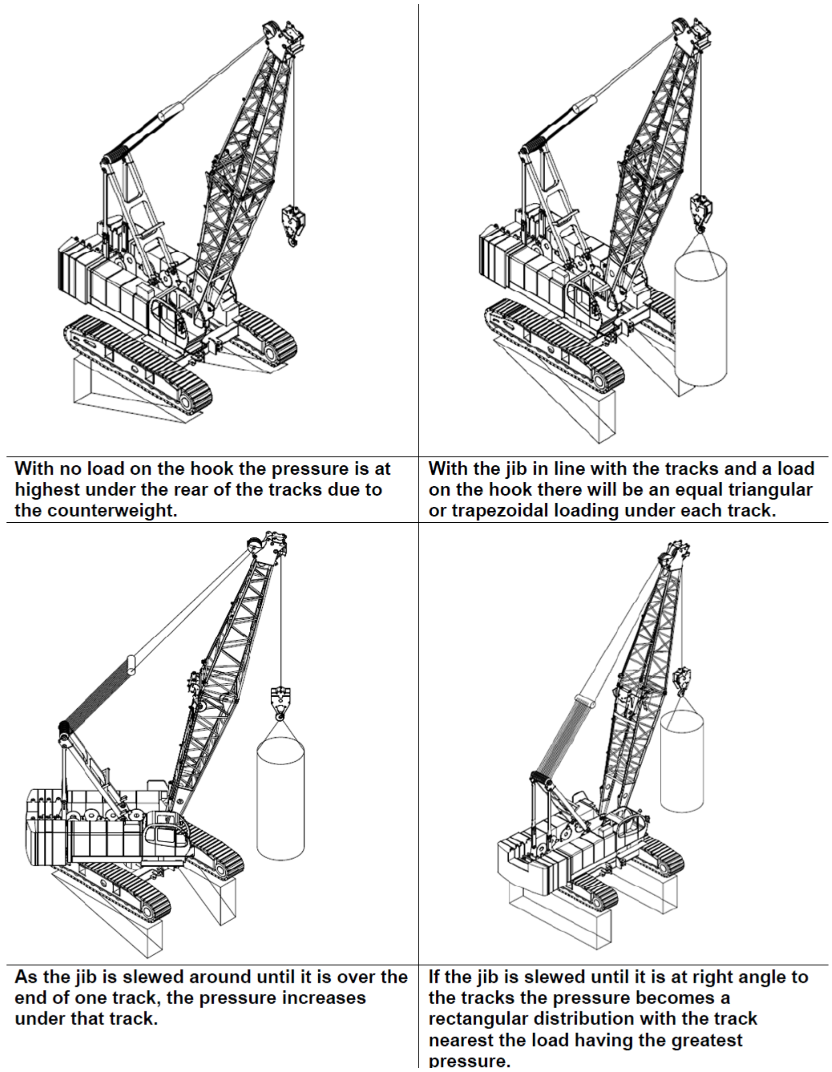
4-6.01 Crane Live Loads

Crane live loads are determined by calculating the forces a crane will exert on the trestle’s deck during operation. As the crane moves and load configurations change, various load cases may arise, with its operation typically restricted by its position on the trestle (see Figure 4-8). It is crucial to identify the load cases that impose the maximum load on the different load-carrying members of the trestle. Ensure the crane operator is fully aware of these restrictions.

The following are some things to consider, when analyzing a crane on a temporary access trestle:

1. Crane specifications: Collect details such as the crane model, boom length, maximum rated load, and configuration (e.g., outrigger spacing for mobile cranes or track bearing dimensions for crawler cranes).
2. Lift details: Identify the weight of the lifted load and the radius (distance from the crane’s center to the load).
3. Outriggers: For cranes with outriggers, compute the static loads on each outrigger based on the load and crane configuration, balancing moments around the crane’s support points.
4. Crawler tracks: Calculate track bearing pressures for crawler cranes by evaluating the total load and its distribution over the tracks.

5. Software utilization: Many crane manufacturers provide ground-bearing pressure estimators or software as standalone tools or website-integrated features to help estimate the loads and pressures a crane will impose based on its configuration and the loads being lifted; see Figure 4-9 for an illustration.



With no load on the hook the pressure is at highest under the rear of the tracks due to the counterweight.

With the jib in line with the tracks and a load on the hook there will be an equal triangular or trapezoidal loading under each track.

As the jib is slewed around until it is over the end of one track, the pressure increases under that track.

If the jib is slewed until it is at right angle to the tracks the pressure becomes a rectangular distribution with the track nearest the load having the greatest pressure.

Figure 4-8. Crawler Track Pressure Change Due to Different Load Cases (Crane Industry Council of Australia)

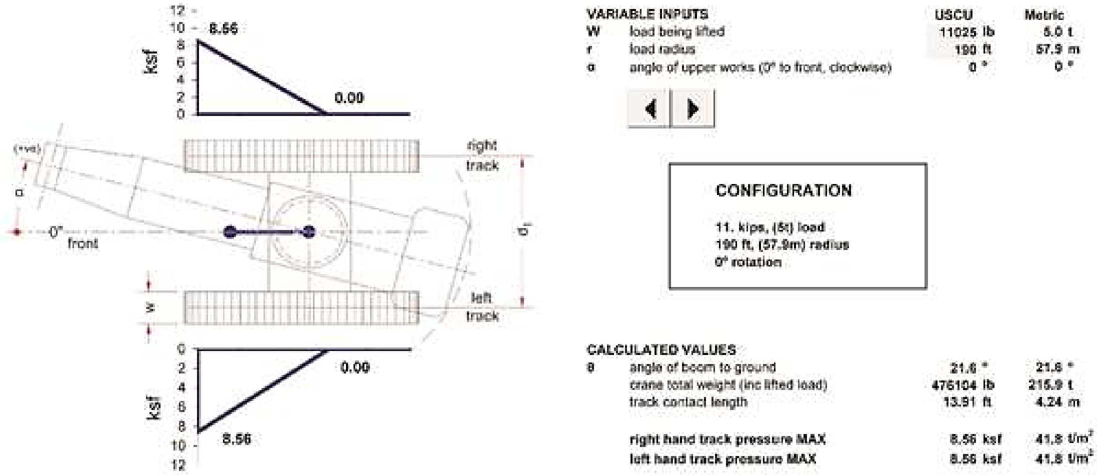


Figure 4-9. Crane Manufacturer's Bearing Pressure Estimator

4-6.02 Dynamic Loads

Various types of dynamic loading to consider on trestles include the following:

1. Vertical impact: Consider whether dynamic forces from acceleration, deceleration, or shifting loads need to be accounted for. These forces are usually minor but should be included for rapid load movements. For fast-moving equipment, live loads may be increased by up to 30 percent to account for these impacts.
2. Horizontal forces: Assess the effect of horizontal forces due to wind, moving water, swinging loads, multi-crane operations, or other equipment.
3. It's crucial to account for all potential loads, including debris buildup. Woody debris accumulating against trestles is a significant concern. Such debris can reduce the capacity of temporary trestles, lead to increased scour, and add extra lateral loading on the pier due to the increased surface area that moving water pushes against.
   a. Figure 4-10 shows an example of debris buildup against a temporary access trestle.
   b. Figure 4-11 is a sketch showing how debris buildup can increase scour (Review Scour Plan of Action for scour critical bridges; contact SM&I Hydraulics Branch for assistance).

Figure 4-10. Debris Buildup Against Trestle

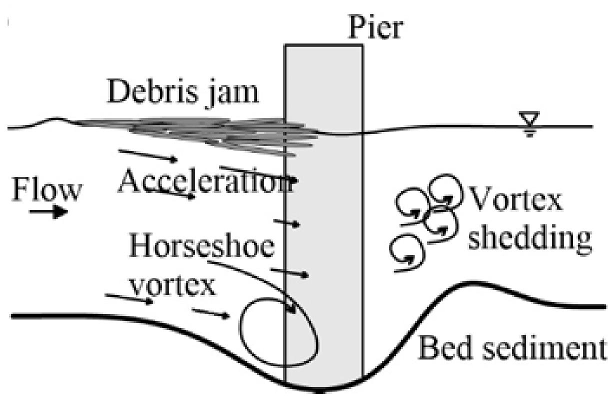


Figure 4-11. Sketch of the Physics of the Scour Process Affected by Debris Buildup

4-6.03 Minimum Lateral Load

The trestle designer will specify this minimum lateral load based on job-specific factors, including equipment types, loads, methods, and other relevant conditions. Typically, the trestle is designed to support a minimum lateral load of 4 to 10 percent of the total vertical load to accommodate dynamic forces from moving live loads.

4-7 Design of Temporary Access Trestles

Due to temporary access trestles having components and materials similar to traditional falsework, guidelines within the Caltrans *Falsework Manual* can be used to determine the trestle's structural adequacy. Like falsework, temporary trestle design typically follows Allowable Stress Design (ASD) principles, ensuring all components stay within the material's elastic range. However, the trestle designer may use any acceptable design method, including Load and Resistance Factor Design (LRFD). The plans should indicate the chosen design method to ensure the reviewer applies the same approach.

Stresses, loadings, deflections, and design values should be based on the most recent editions of the *National Design Specification (NDS) for Wood Construction* and the *American Institute of Steel Construction (AISC) Steel Construction Manual*.

Additionally, temporary access trestles situated over or near railroads must adhere to current railroad safety guidelines.

Figure 4-12. Trestle Used at Golden Gate Bridge, c.1934

Chapter 5: Guying Forms and Rebar Assemblies

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5-1 Introduction

Guying systems are used to support structure components until they become self-supporting. During column construction, guying systems are used to support column cage rebar and column forms. Guying systems are also used to support forms and rebar assemblies for walls and other structural components. Guying systems typically consist of either wire rope (guy wire) for a tension-only system (Figure 5-1) or rigid struts (braces) for a push-pull system (Figure 5-2). The guying system is typically secured to a concrete block deadman or other load-resisting member to resist the applied loads.

A properly designed system with the appropriate installation sequence and procedure, constructed in accordance with the authorized shop drawings, will ensure that the column rebar and column forms will not collapse during construction. A column collapse has many adverse results which include equipment and property damage, injuries, schedule delays, and expensive repair costs.

Figure 5-1. Guy Wire System, Butte City Bridge

Figure 5-2. Support Frame System, Antlers Bridge

5-2 Contractual Requirements

The *Contract Specifications* Section 52-1.01C(2), *Reinforcement – General – Submittals – Shop Drawings,* requires the Contractor to submit to the Engineer for review and authorization, temporary support system shop drawings and calculations. This temporary support system plan is commonly referred to as a “guying plan”.

Submittal of the temporary support system plan is required if a portion of an assemblage of bar reinforcing steel exceeds 20 feet in height and is not encased in concrete. Forms that encase the rebar must also be secured to resist overturning.

Temporary support system shop drawings and calculations must be sealed and signed by an engineer who is registered as a civil engineer in the State.

5-3 Cal/OSHA Requirements

Requirements from Cal/OSHA's Construction Safety Orders, Article 29, *Erection and Construction*:

§ 1711, *Reinforcing Steel and Post-Tensioning in Concrete Construction*

(a) 1711(e)(1) states: "Reinforcing steel for walls, piers, columns, prefabricated reinforcing steel assemblies, and similar vertical structures shall be guyed, braced, or supported to prevent collapse."

(a) 1711(e)(3) states: "Reinforcing steel shall not be used as a guy or brace."

§ 1713, *Framed Panels and Concrete Forms*

(a) 1713(b) states: "Form panels for concrete structures shall be securely anchored, guyed, or braced to prevent them from falling or collapsing."

5-4 Review and Authorization

Guying systems are temporary structures and are reviewed and authorized in accordance with BCM C-11, *Shop Drawing Review of Temporary Structures.*

Initial review of the submittal for completeness should check for the following items:

1. Legible drawings
2. Stamped by a registered civil engineer in the State.
3. All components identified and information for manufactured assemblies included.
4. Supporting calculations
5. All dimensions shown on the shop drawings.
6. Sequence and installation/removal procedures included.
7. Anchors and cables not conflicting with existing roadways/structures/utilities.

An independent analysis by Caltrans should be performed to verify whether the capacity of the guy wire system components is greater than the applied wind load against the column assemblage. See Appendix A Example 2*, Column Guying*, for an example of a column guying analysis.

A detailed work plan is required for guying submittals that involve the railroad. The work plan must include procedures for installing rebar assemblies and forms. The work plan must include the following:

1. Crane pick plan for rebar assemblies and forms.
2. Procedures for installing forms without releasing supports.

3. Staging areas for equipment and materials.
4. Path of travel for equipment in and out of railroad right-of-way.
5. Utilities within the railroad right-of-way.
6. Critical dimensions including dimensions to centerline of track.

5-5 Sequence and Installation Procedures

Special emphasis should be placed on the requirement of *Contract Specifications* Section 52-1.01C(2), *Shop Drawings*, which states, ***"****If form installation or other work requires changes to or the temporary release of any part of the temporary support system, the shop drawings must show the support system to be used during these changes or the temporary release.****"*** On a fixed column-to-footing connection, the column rebar cage is imbedded in the footing. A column guy wire system typically supports the column rebar cage during the footing construction. After the footing concrete is placed, the column forms are installed. However, the guy wire locations usually conflict with the column forms and must be temporarily removed. During this stage, the column rebar cage is vulnerable to instability and collapse. It is very important that the Contractor has a sequence and installation procedure. The sequence and installation procedure can be written as general notes on the drawings, or it can be on a separate sheet, which is included in the submittal.

Typically, for a fixed column, two cranes are needed. The Contractor will use one crane to support the column rebar cage while the guy wire system is temporarily removed. The second crane will be used to install the column forms, and then the guy wire system will be reinstalled.

Typically, for a pinned/hinged column, the Contractor will use a crane to erect the column forms. Then the guy wire system will be installed on the column forms. After the guy wire system is installed, the same crane will be used to lift the rebar cage and set it inside the column form.

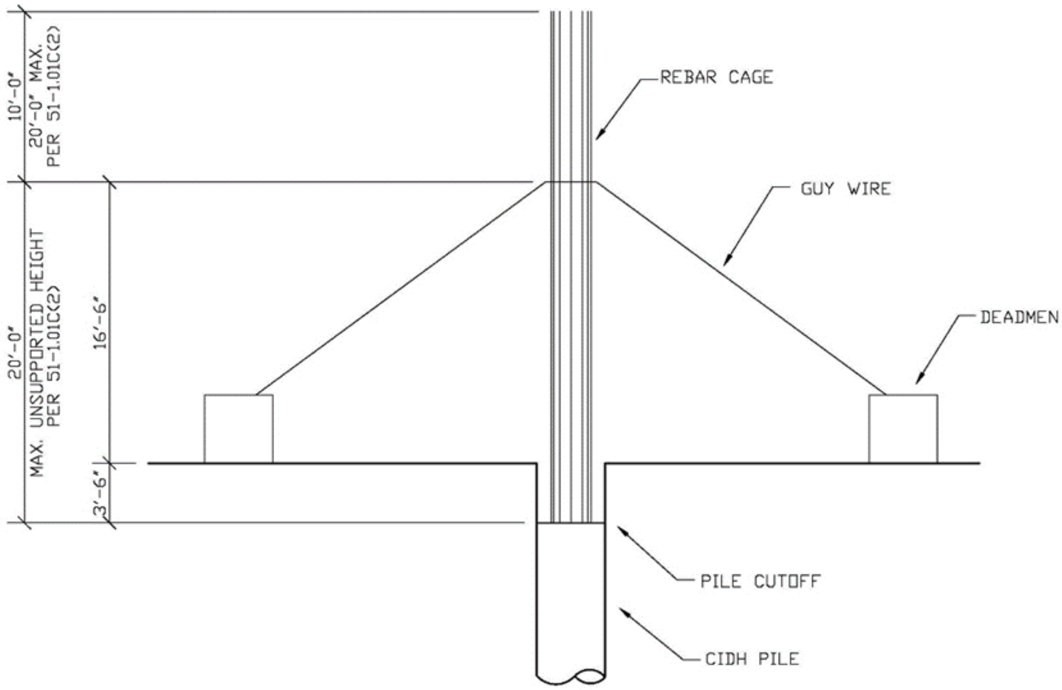

Figure 5-3. Rebar Assembly Support

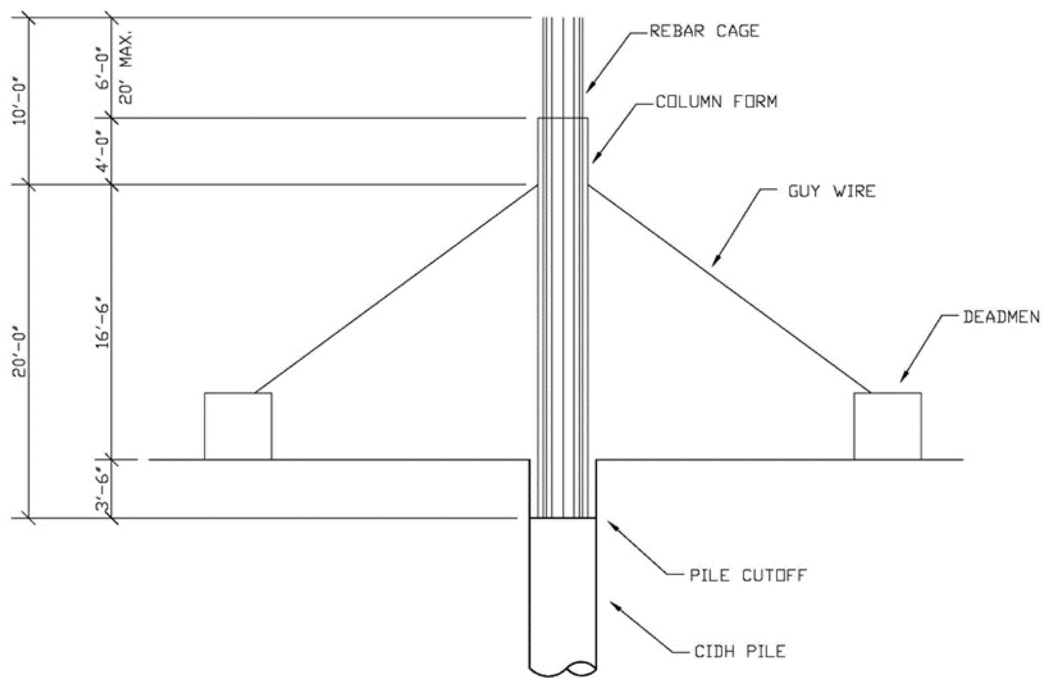

Figure 5-4. Column Form Support

5-6 Guying System Design

The minimum horizontal wind load to be applied to the reinforcing steel assemblage or to a combined assemblage of reinforcing steel and forms must be the sum of the products of the wind impact area and the applicable wind pressure value for each height zone. Wind pressures for forms and rebar assemblages are found in the previously referenced *Contract Specification* Section 52-1.01C(2), S*hop Drawings,* as shown in Figure 5-5 below:

Wind Pressure

Height zone, H (feet above ground)	Wind pressure value (psf)
0 ≤ 30	20
30 < H ≤ 50	25
50 < H ≤ 100	30
H > 100	35

Figure 5-5. Wind Pressure Height Zones

The wind impact area is the projected area of the forms or rebar assemblage normal to the direction of the applied wind. The projected area of rebar assemblages is the full area and includes openings through the reinforcement. Reinforcement that cantilevers above the rebar cage is also included, and the areas between the rebar are included.

Cable bracing for guying systems should be designed using the procedures found in the *Falsework Manual* Chapter 5, *Analysis*, Section 5-5, *Cable Bracing Systems*. A minimum factor of safety, **FS = 2**, based on the minimum breaking force, is required when determining the allowable design capacity of the cable units.

In most cases, cables will be secured by fastening the end to a concrete anchor block (deadmen), although temporary cast-in-drilled hole (CIDH) anchors are sometimes used when relatively large forces must be resisted.

For either concrete anchor blocks or CIDH anchors, the method of connecting the cable to the anchorage is part of the design. The connecting device must be designed to resist both vertical (uplift) and horizontal forces. If manufactured devices are used to connect the guy cable, follow the manufacturer’s instructions for loadings.

For the procedure to review cable anchored to CIDH anchors, see *Falsework Manual* Section 5-6, *Short Poured-In-Place Concrete Piles.*

Concrete anchor blocks must be proportioned to resist both sliding and overturning. The weight of the anchor block must be reduced by the vertical component of the cable tension to obtain the net or effective weight to use in the anchorage computations.

For dry service conditions:

The coefficient of friction assumed between the concrete anchor block and base material must not exceed the values displayed in Figure 5-6.

For wet service conditions:

Multiply the values for dry conditions by 0.67. This reduction must be used if it is likely that the base material will become wet (saturated) during the construction period.

If the blocks are submerged, buoyancy effects must be addressed.

Friction of Concrete Anchor Blocks	
Base Material	**Coefficient of Friction**
Sand	0.40
Clay	0.50
Gravel	0.60
Pavement	0.60

Figure 5-6. Industry Standard Coefficient of Friction for Concrete Anchor Blocks

The minimum factor of safety for overturning and sliding of deadman anchor blocks is one (**FS = 1**) in each case.

Chapter 6: Deck Plates

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6-1 Introduction

Temporary deck plates in accordance with *Contract Specifications* Section 48-4, *Temporary Structures – Temporary Decking*, provide the means of temporary decking for bridge joint or deck reconstruction. Deck plates may be considered as an option if bridge reconstruction activities are unable to be completed before the bridge is to be opened to traffic. The design of deck plates typically consists of the use of metal plates mechanically anchored to the existing deck or approach slab, while tapering the plate edges to the traveling surface for a smooth transition. Note that tapering of the plates may not be required if the plates are recessed, as illustrated in Figure 6-1.

Deck plates may be a Caltrans-designed system, as shown in the contract documents, or a contractor-designed system when not shown in the contract documents.

Figure 6-1. Recessed Deck Plates Installed for Deck Joint Replacement Sacramento River BOH, Bryte Bend (Br. NO. 22-0026). Route: 80

6-2 Contractual Requirements

The *Contract Specifications* Section 48-4, *Temporary Structures – Temporary Decking,* requires the Contractor to submit to the Engineer for authorization, temporary decking shop drawings and calculations. The shop drawings must be sealed and signed by an engineer who is registered as a civil engineer in the State.

6-3 Design Considerations

If temporary decking is not shown, the temporary decking design must:

1. Comply with the unfactored permit loads, braking force, and HL93 loads, except lane load, from the current *AASHTO LRFD Bridge Design Specifications with California Amendments*.
2. Not exceed the allowable stresses or design loads specified in *Contract Specifications* Section 48-2.02B(3), *Temporary Structures – Falsework – Materials – Design Criteria – Stresses, Loadings, and Deflections*.
3. Have live load deflection not exceeding 1/300 of the temporary decking span length for the design load.
4. Provide for temporary decking with a uniform surface and a coefficient of friction of at least 0.35 when measured under California Test 342, *Method of Test for Surface Skid Resistance with the California Portable Skid Test*.
5. Provide for temporary decking that is mechanically connected to the existing structure and adjacent approaches. If a steel plate spans a joint, the mechanical connection must accommodate at least 50 percent of the movement range shown for that joint.
6. Not overstress, induce permanent forces into, or produce cracking in the existing structure.

If there is an elevation difference of more than 1/2 inch between the temporary decking and the adjacent deck, install temporary tapers up to and away from the temporary decking. Construct tapers under *Contract Specifications* Section 7-1.03, *Legal Relations and Responsibility to the Public – Public Convenience.* If the temporary decking does not extend the entire width of the roadway, taper the sides of the temporary decking at a 12:1 (horizontal: vertical) ratio. Material for temporary tapers must comply with *Contract Specifications* Section 60-3.02B(2), *Existing Structures – Structure Rehabilitation – Bridge Deck Repair or Preparation – Materials – Rapid Setting Concrete*, or Section 60-3.04B(2), *Deck Overlays – Polyester Concrete Overlays – Materials*. Cure temporary tapers at least 3 hours before allowing traffic on the temporary decking.

6-4 Caltrans-Designed Deck Plates

Caltrans-designed temporary deck plates are shown in the contract documents. Refer to the project plan sheets and the special provisions for unique requirements.

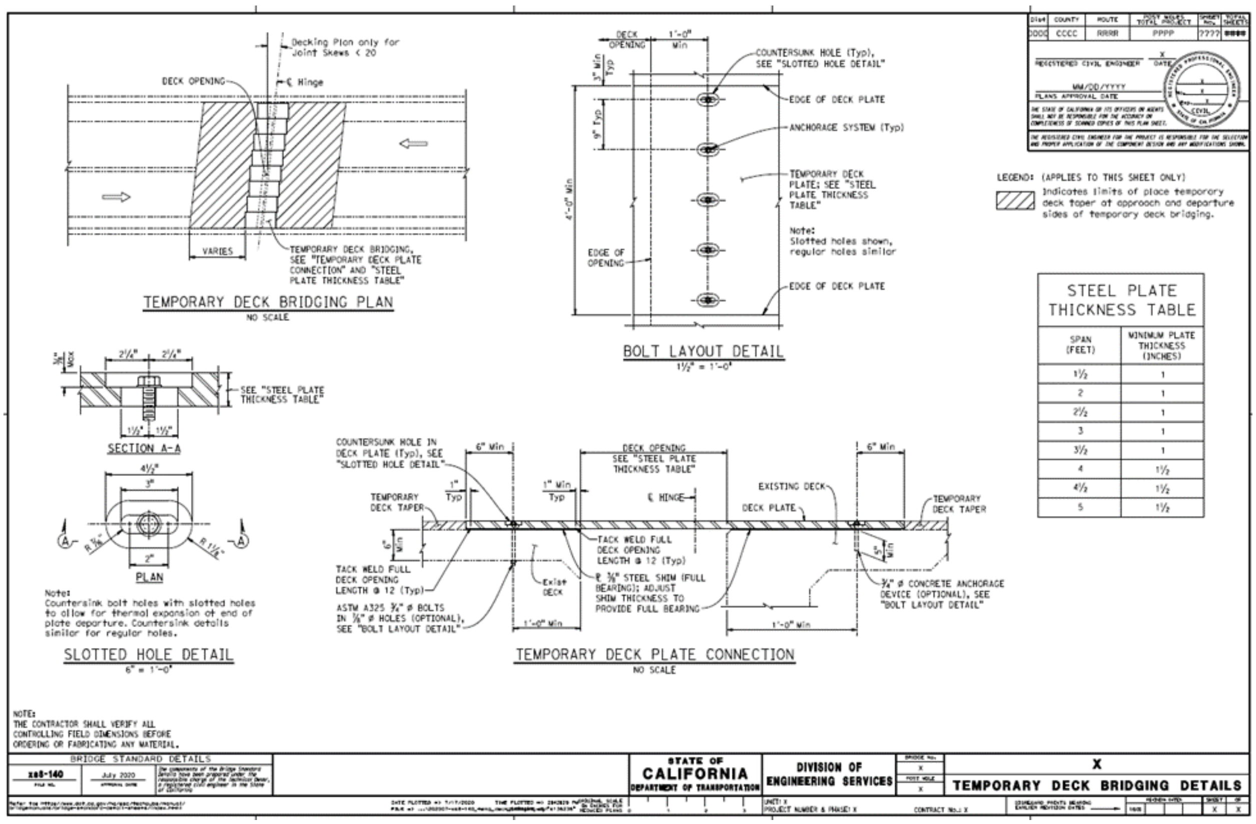

Figure 6-2. Contract Plan Temporary Deck Plate Details

Restrictions on Use of Standard Drawings:

The maximum longitudinal clear span is 5 feet. Spans greater than 5 feet may require a special design to control the deflection. The required plate thickness may create a bump greater than what District Traffic Engineering will allow. The maximum joint skew for Temporary Deck Bridging XS Sheet is 20 degrees. Greater skews create grade problems with the temporary taper and plate stagger. To solve this, the plates are placed parallel to the joint. The exterior plates are cut in a triangle, which requires special detailing.

6-5 Contractor-Designed Deck Plates

The Contractor is responsible for the design of the temporary deck plates if the contract documents do not include Caltrans deck plate drawings.

Contractor-designed deck plates must be submitted to the Engineer for review and authorization under *Contract Specifications* Section 48-4.01C, *Temporary Structures – Temporary Decking – Submittals.* The Contractor is required to submit shop drawings and calculations for temporary decking, which must be sealed and signed by an engineer who is registered as a civil engineer in the State.

Refer to the aforementioned specifications for specific requirements for storage location of equipment and materials, construction sequence and schedule, cure time for concrete, and removal details for the temporary decking.

For temporary decking which is not shown in the plans, shop drawings, and calculations must also include:

1. Design calculations, including the description, location, and value of all loads.
2. Details of the connection between the temporary decking and the existing or new structure, including range of movement.

A certificate of compliance for temporary decking materials must be submitted.

6-6 Installation, Maintenance, and Removal

The Contractor is expected to inspect the installed temporary deck plates daily while in service, where damage may occur due to wear and tear. Components of inspection include the taper, metal plate, and anchorage system.

Some common maintenance repairs include:

1. Any loose hardware of the anchorage system shall be securely tightened.
2. Verify all hardware is in place per plan; the Contractor should have additional hardware on-hand if needed.
3. Any visible vibration on plates with loading is required to be securely tightened with an impact and shims checked for full bearing.
4. Shims may need to be replaced if plate does not maintain appropriate height.
5. Missing or damaged shims require replacement; the Contractor must have additional shims on hand to add/replace as needed.
6. Patch any damage to the taper if cracking or spalling occurs.

7. Replace any missing foam between plate and taper as needed.

Installation and maintenance checklists are useful tools to ensure deck plates are installed and maintained properly. See Figure 6-3 and Figure 6-4 below for examples of deck plate installation and daily inspection checklists.

Discuss the use of these checklists at the preconstruction conference.

As outlined in *Contract Specifications* Section 48-4.03, *Temporary Structures – Temporary Decking – Construction*, verify that the Contractor promptly removes the temporary decking when no longer needed, the holes are patched, and the existing structure is left in the condition required.

Temporary Deck Plates Installation Checklist

Date: ______________________________

Location: ______________________________

Inspector: ______________________________

☐ Steel plate thickness matches shop drawings

☐ Plates sit flush on deck (full bearing)

☐ Plates centered correctly (even spacing of demolition limits)

☐ Shims/crush strips under plates positioned correctly

☐ High spots ground to provide full bearing (no rocking)

☐ Slotted and fixed bolt holes oriented correctly, to allow for thermal expansion

☐ Plates installed per installation plan

☐ Shims in between the plates placed correctly

☐ Anchor holes drilled using the correct diameter drill bit

☐ Anchor holes drilled to the correct depth

☐ Anchor holes cleaned per manufacturer instructions

☐ Anchors embedded to the minimum depth specified in shop drawings

☐ Anchors installed per manufacturer's instructions

☐ Anchors match what is specified in the shop drawings

☐ Rapid strength or polyester concrete tapers installed per authorized shop drawings

☐ Foam strips are placed per authorized shop drawings

Figure 6-3. Example Deck Plate Installation Checklist

Date: ______________________________
Plates installed: ______________________________
Location (Joint#/Bent #) ______________________________
Inspector: ______________________________

Temporary Deck Plates Daily Inspection Protocol

On a daily basis, each taper, roadway plate, and anchorage system installed will be inspected for the following:

1. Check if hardware is loose - securely tighten as needed
2. Verify replacement nuts and washers are available onsite
3. Check for visible movement of anchors - replace as required
4. Check for excessive vibration of plates - tighten anchors as needed
5. Verify shims maintaining appropriate height – replace if flattened
6. Verify shims are oriented correctly – replace as needed
7. Check for cracking or spalling in tapers – patch damaged tapers
8. Verify foam in place between plate and taper – replace as needed

Figure 6-4. Example Deck Plate Daily Inspection Checklist

Chapter 7: Bridge Scaffolding

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7-1 Introduction

This chapter addresses bridge scaffolding defined by *Contract Specifications* Section 7-1.02K(6)(e), *Legal Relations and Responsibility to the Public – Laws – Labor Code – Occupational Safety and Health Standards – Scaffolding*, as scaffolding constructed on or suspended from a bridge. Bridge scaffolding provides temporary access to areas of the bridge for workers and materials during bridge repair, retrofit, painting, and other construction activities. Bridge scaffolding is typically a contractor designed system or a proprietary system designed by a third party. Bridge scaffolding can take many forms from traditional scaffolding rigid brackets (Figure 7-1) mounted to the structure, to flexible systems hung by cables below the bridge (Figures 7-2 and 7-3). Walkways for falsework systems are not considered a scaffold system.

Figure 7-1. Conventional Scaffold System Supported by Structure

Figure 7-2. Suspended Scaffold System - Quikdeck

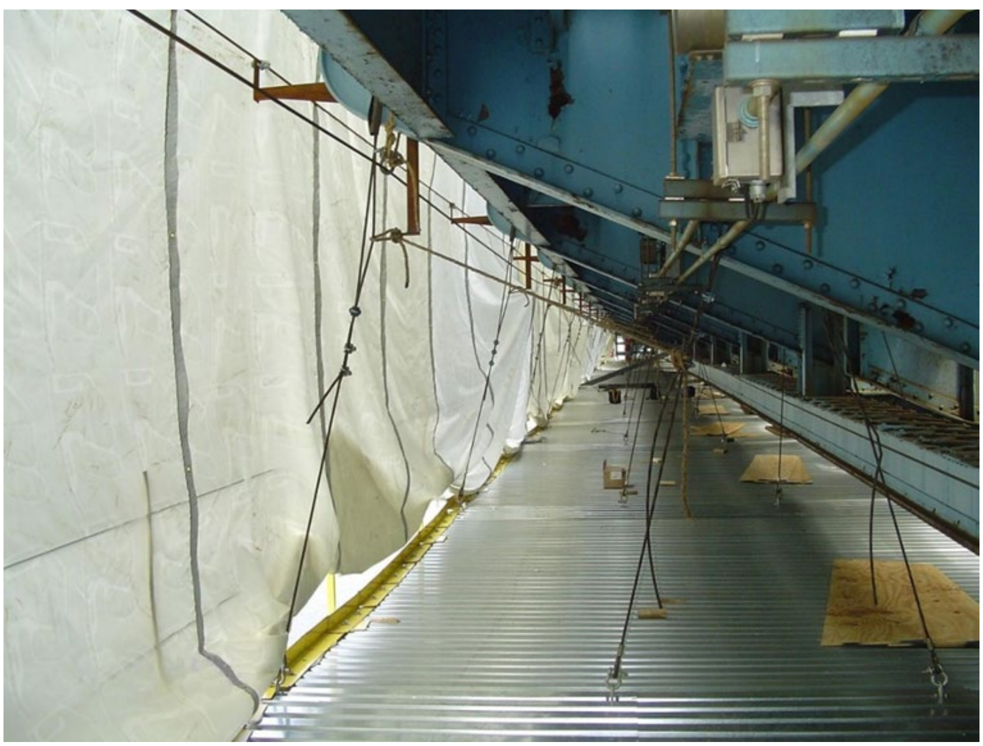

Figure 7-3. Suspended Scaffold System - Safespan

7-2 Contractual Requirements

The *Contract Specifications* Section 7-1.02K(6)(e), *Scaffolding,* requires the Contractor to submit to the Engineer for review and authorization, scaffolding shop drawings and calculations. In addition, an independent check is required by an engineer not employed by the same entity that prepared the drawings.

The scaffolding shop drawings and calculations, along with the independent check, must be sealed and signed by an engineer who is registered as a civil engineer in the State.

The available bridge uniform load, in pounds per square foot (psf), is the available bridge load capacity in excess of the vehicular live load and is typically found in Section 7-1.02K(6)(e), *Scaffolding,* of the *Contract Specifications*; refer to Figure 7-4 for an example. The available bridge uniform load is used to calculate the available shear and moment capacity envelopes. The shear and moment demand envelopes are determined using the actual scaffold loads. The live load increases required by Cal/OSHA are not considered when determining the demand on the existing structure. If the demand envelope for each bridge member is less than the available capacity envelope, the proposed scaffold system is acceptable for the global check of the existing structure. Investigation of member stresses is not required, provided the applied loading is compared to the available bridge load capacity.

1. Aiken Creek Bridge (br. no. 04-0059) and Slate Creek Bridge (br. no. 04-0061):

Bridge name/number	Available bridge load capacity (lb/sq ft)	Bridge width (ft)	HL-93 design live load			Permit design live load	
			Percentage of HL-93 loading (percent)	No. LL_{HS20} lanes		Permit vehicle	No. LL_{Permit} lanes
				Moment[b]	Shear		
Aiken Creek Bridge (br. no. 04-0059)	65	29.5	81	1.03 for exterior girders 0.99 for interior girders	0.64 for exterior girders 0.74 for interior girders	N/A	N/A
Slate Creek Bridge (br. no. 04-0061)[a]	65	29.5	70	0.64 for exterior girders 0.60 for interior girders	0.64 for exterior girders 0.67 for interior girders	N/A	N/A

[a] Denotes live load is only permitted on half of the structure at Slate Creek Bridge (br. no. 04-0061)
[b] Denotes Live Load Distribution Factors (LLDF) varies along the length of structures. The values provided in the table are for the controlling locations only. All the other locations to be verified by the Contractor.

Figure 7-4. Example Available Load Capacity Table

The adequacy of the existing bridge components at or near points of support must also be investigated. The *Contract Specifications* requires the evaluation of existing members be performed using the *AASHTO LRFD Bridge Design Specifications with California Amendments*, latest edition.

The Contractor can elect to do a more detailed analysis by not simply comparing the shear and moment envelopes using the available bridge uniform load as described above. A more detailed analysis is performed using the percentage of HL-93 live loading along with maximum number of live load HL-93 lanes given in the *Contract Specifications* Section 7-1.02K(6)(e) *Scaffolding*. The detailed analysis would be based on the *AASHTO LRFD Bridge Design Specifications with California Amendments*, latest edition. Specific load modifiers for performing the analysis are given in the aforementioned Section 7-1.02K(6)(e).

For truss type bridges, if the proposed shop drawings do not comply with the scaffold staging shown in the contract documents, the calculations must also include tension and compression force demand versus capacity of truss members during erection, movement, and removal of the scaffold. All connection must be made through stringers, floor beams, or truss panels. Connections that may cause bending stresses in a truss member are not allowed.

7-3 Cal/OSHA Requirements

Requirements from Cal/OSHA's Construction Safety Orders:

Article 21, *Scaffolds – General Requirements*, § 1637, *General Requirements,* subsection 1637(b), *Scaffold Design and Construction,* requires scaffolding to be designed and constructed using a dead load safety factor that will ensure the scaffold supports, without failure, its own weight and 4 times the maximum intended working (live) load applied or transmitted to it. Light duty scaffolds shall have a maximum working load of 25 psf, medium duty scaffold a maximum working load of 50 psf, and heavy duty scaffold a maximum working load of 75 psf.

Requirements for specific types of scaffolds can be found in Article 22, *Scaffolds – Various Types*, as follows:

§ 1644, *Metal Scaffolds*

§ 1645, *Outrigger and Bracket Scaffolds*

§ 1646, *Tower and Rolling Scaffolds, Wood or Metal.*

7-4 Review and Authorization

Scaffolds are a temporary structure and are reviewed and authorized in accordance with BCM C-11, *Shop Drawing Review of Temporary Structures.*

Initial review of the submittal for completeness should check for the following items:

1. Legible drawings
2. Stamped by a registered civil engineer in the State.
3. Independent check stamped by a registered civil engineer in the State.
4. All components identified and information for manufactured assemblies is included.
5. All dimensions shown on the shop drawings.
6. Sequence and installation/removal procedures.
7. Calculations for scaffold system and existing structure.
8. Description and values for scaffold loads during erection, movement, and removal.

An independent analysis by Caltrans (as described in Section 7-5, *Resources*) should be performed to verify the additional loading on the existing bridge structure is within the shear and moment envelopes specified in the contract documents. The scaffold system and the connections to the existing structure must also be reviewed and authorized by the Engineer. Portions of the scaffold system that are proprietary can be treated as manufactured assemblies and the allowable loads determined by adhering to the loading instructions supplied by the manufacturer.

A detailed work plan is required for scaffold submittals that involve the railroad. The work plan must include:

1. Procedures for installing, maintaining, and removing the scaffold system.
2. Path of travel for equipment in and out of railroad right-of-way.
3. Critical dimensions including horizontal and vertical dimensions to centerline of track measured from top of rail elevation.
4. Staging areas for equipment and material.
5. Equipment to be used including location and swing path.

See Appendix A Example 3, *Bridge Scaffolding,* for typical scaffold review.

7-5 Resources

The analysis of the existing bridge components using Load and Resistance Factor Design (LRFD) and Load Factor Design (LFD) may require the assistance of the Bridge Design Professional Engineer for Bridge Design projects. If the project is a Structure Maintenance and Investigation (SM&I) project, then SM&I would provide support. Software not available to SC field staff might also be required to properly analyze the existing structure. The SM&I or the Bridge Design project engineer should be contacted early in the development of the scaffold shop drawings so critical issues can be addressed.

The SC Falsework Engineer is another resource when doing the independent analysis and is the point of contact when the railroad is involved.

Chapter 8: Temporary Bridges

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8-1 Introduction

This chapter addresses temporary bridge structures that are either designed by the Contractor or included in the contract documents. Temporary bridges play a crucial role in maintaining the continuous flow of the transportation system and ensuring the safety of the traveling public. These structures are designed and installed to provide a reliable crossing over roadways, waterways, or other obstacles when the permanent bridge is unavailable due to damage, maintenance, construction, or emergencies like accidents or natural disasters.

A typical section of a temporary bridge is illustrated in Figure 8-1, and a temporary modular steel truss bridge is pictured in Figure 8-2.

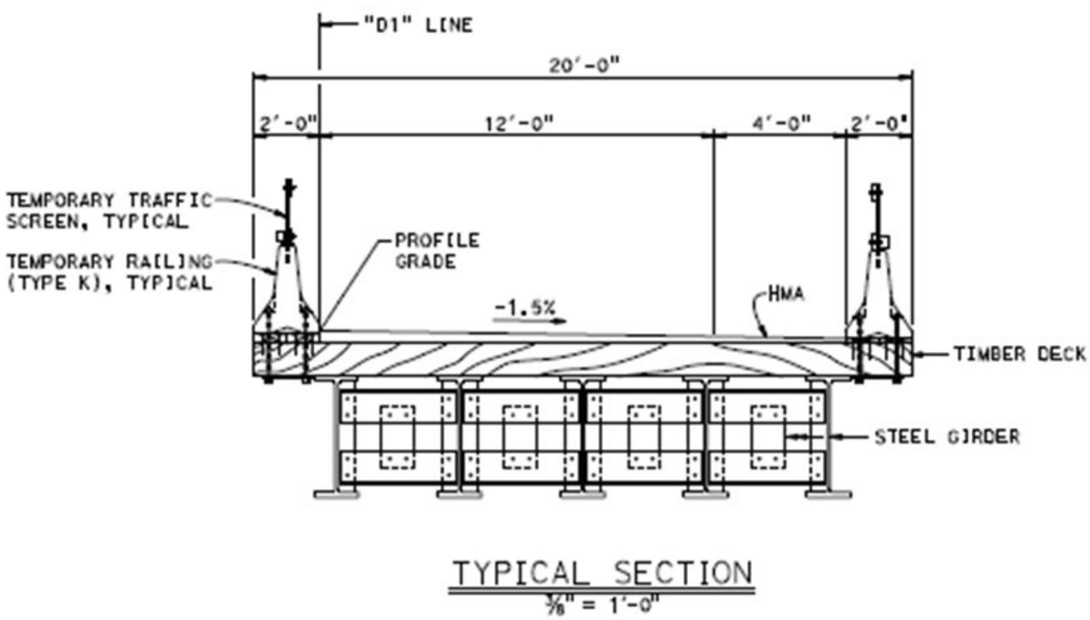

Figure 8-1. Typical Temporary Bridge Section

Figure 8-2. Temporary Modular Steel Truss Bridge – Capell Creek

8-2 Contractual Requirements

Temporary bridges are either specified in the contract documents or proposed by the Contractor. If specified in the contract documents, then the special provisions and project plans will have the requirements for the temporary bridge. Some contracts require a quality control (QC) meeting for the temporary bridge before submitting QC and erection plans. If a temporary bridge was not anticipated during the design phase and the Contractor proposes a temporary bridge for their means and methods, then a change order will need to be written with the requirements for the temporary bridge.

Temporary bridge shop drawings and design calculations must be sealed and signed by an engineer who is registered as a civil engineer in the State. When required by the *Contract Specifications*, independently checked calculations must be prepared, sealed, and signed by an engineer registered as a civil or structural engineer in the State.

The design and construction of the temporary bridge superstructure and substructure components must comply with the requirements of current *AASHTO LRFD Bridge Design Specifications with California Amendments*.

It is the Contractor's responsibility to build, maintain, and remove the temporary bridge structure along with the approaches to the temporary bridge.

8-3 Loads

To follow are loads and considerations for the design of temporary structures:

Service Loads: *Memo to Designers* 15-14, Attachment 1, *Loads for Temporary Highway Structures*, includes loads for temporary highway structures.

Extreme Event Limit State: Seismic design of temporary bridges is required in accordance with *Memo to Designers* 20-2, *Site Seismicity for Temporary Bridges and Stage Construction*.

Temporary Prefabricated Modular Steel Panel Truss Bridges: The requirements for the design of temporary modular steel truss bridges are provided in the *Structure Technical Policy* 17.1, *Design Criteria for Temporary Prefabricated Modular Steel Panel Truss Bridges*.

8-4 Review and Authorization

When the temporary bridge is designed by the Contractor, the shop drawings and design calculations are to be submitted directly to Structure Construction Office Associates (sc.office.associates@dot.ca.gov) and then routed to Bridge Design for review and approval. Shop drawings and design calculations must be sealed and signed by a registered engineer in the State. Details and information that must be included in the shop drawings can be found in the contract documents.

Temporary modular steel bridges designed by the Contractor must be accompanied and supported by independently checked calculations. The independently checked calculations must be prepared, sealed, and signed by an engineer who is (1) not employed by the modular bridge manufacturer and (2) registered as a civil or structural engineer in the State.

The Structure Representative (SR) will review the temporary bridge submittal for completeness and will review portions of the design submittal, such as temporary shoring walls for abutments, that are not reviewed by Bridge Design. The SR will authorize the temporary bridge submittal after receiving approval from Bridge Design and other stakeholders.

8-5 Quality Control Plan

When required by the *Contract Specifications*, the Contractor must submit a quality control (QC) plan for the temporary bridge. The requirements for the QC plan can be found in the special provisions.

8-6 Erection and Removal Plan

Erecting and removing the temporary bridge must be in accordance with the authorized temporary bridge erection and removal plan. The erection and removal plan must be sealed and signed by a registered engineer in the State, and plan requirements can be found in the contract documents.

An erection and removal report must be prepared for each day any erection or removal activities are being performed for modular steel bridges. The report must describe and document all activities and findings. Check the *Contract Specifications* to determine which temporary bridges require this report.

8-7 Material Documentation

Materials used in constructing the temporary bridge must be submitted in accordance with the contract documents. Although not a comprehensive list, material information that must be submitted includes the following:

1. List of any used materials and where the materials will be incorporated in the temporary bridge, on Form CEM-3101, *Notice of Materials to be Used*.
2. Certificate of compliance for the members and fabrication of the modular steel bridge.

All modular steel bridge members, including pins and fasteners, must be listed on Form CEM-3101, *Notice of Materials to be Used*.

8-8 Maintenance

The Contractor is responsible for performing maintenance inspections as required by the contract documents and submitting the required inspection reports to the Department. Inspection frequency, reporting requirements, and corrective action can be found in the contract documents.

Chapter 9: Temporary Supports and Jacking

Table of Contents

9-1 Introduction

Temporary supports are used to support structures during retrofit, reconstruction, erection, and removal activities. The design of temporary supports is similar to falsework except for some unique loading and design requirements. One major difference between falsework and temporary supports is the duration of the maximum loading. Falsework is typically loaded with plastic concrete for only a day or two, but temporary supports can be in place for months. Temporary supports are often associated with jacking operations which will be discussed in this chapter. Jacking is done to raise or lower structures or just to relieve the load for repairs. See Figure 9-1 for support of precast girders, see Figure 9-2 for an illustration of lowering a bridge, and Figure 9-3 for an illustration of raising a bridge.

Figure 9-1. Route 46/99 Separation ABC Project

Figure 9-2. Temporary Support for Bridge Lowering

Figure 9-3. Temporary Support Highway 70 Bridge Raising

9-2 Contractual Requirements

The *Contract Specifications* Section 48-3, *Temporary Structures – Temporary Supports,* requires the Contractor to submit to the Engineer for review and authorization, temporary support shop drawings and calculations. Temporary support shop drawings and calculations must be sealed and signed by an engineer who is registered as a civil engineer in the State. Specific design criteria can be found in the *Contract Specifications* or shown on the project plans.

Jacking is typically associated with temporary supports, and the requirements are found in the *Contract Specifications* Section 48-5, *Temporary Structures – Jacking*. The Contractor must submit shop drawings and calculations, which include the information listed in Section 48-5 for the jacking system. Jacking system shop drawings and calculations must be sealed and signed by an engineer who is registered as a civil engineer in the State.

Contract Specifications Section 48-5.01D(2) *Temporary Structures – Jacking – General – Quality Assurance – Displacement Monitoring*, requires control points on the structure be monitored at a minimum as follows:

1. Before starting jacking activities.
2. Immediately after completing jacking.
3. After completing bridge removal.
4. Before connecting the superstructure to the substructure.
5. After removing the jacking support system.

The displacement monitoring plan should either be a separate submittal or part of the jacking submittal.

9-3 Cal/OSHA Requirements

Requirements from Cal/OSHA's Construction Safety Orders:

Article 29, *Erection and Construction*, § 1717, *Falsework and Vertical Shoring,* subsection 1717(b)(1), *Design,* requires [falsework or] vertical shoring be designed by a civil engineer, currently registered in the State, when any of the following conditions exist:

(A) The height, as measured from the top of the sills to the soffit of the superstructure, exceeds 14 feet

(B) Individual horizontal span lengths exceed 16 feet

(C) Provisions for vehicular or railroad traffic through the falsework or vertical shoring are made.

§ 1717(c), *Inspection,* subsection (1), requires:

After construction of the falsework or vertical shoring system enumerated in section 1717(b)(1) and prior to placement of concrete, a civil engineer, currently registered in California, or authorized representative, shall inspect the falsework or vertical shoring system for conformity with the working drawings. The person performing the inspection shall certify in writing that the falsework or vertical shoring system substantially conforms to the working drawings and that the material and workmanship are satisfactory.

9-4 Review and Authorization

Temporary support and jacking systems are temporary structures and are reviewed and authorized in accordance with BCM C-11, *Shop Drawing Review of Temporary Structures.*

Initial review of the submittal for completeness should check for the following items:

1. Legible drawings
2. Stamped by a registered civil engineer in the State
3. Supporting calculations
4. All components identified and information for manufactured assemblies are included.
5. All dimensions shown on the shop drawings.
6. Sequence and installation procedures are included.

An independent analysis by Caltrans (typically SC staff) should be performed to verify if the design of the temporary support and jacking system conforms to the *Contract Specifications* and stresses in members are within the specified allowable limits. Documentation for calibration of the jacks and gauges should be reviewed for conformance with the *Contract Specifications*.

A detailed work plan is required for guying submittals that involve the railroad. The work plan must include procedures for installing rebar assemblies and forms. The work plan must include the following:

1. Crane pick plan for rebar assemblies and forms.
2. Procedures for installing forms without releasing supports.
3. Staging areas for equipment and materials.

4. Path of travel for equipment in and out of railroad right-of-way.
5. Utilities within the railroad right-of-way.
6. Critical dimensions including dimensions to centerline of track.

9-5 Temporary Support System Design

Temporary support systems have unique requirements that differ from typical falsework design. The most notable difference is that the temporary support design requires the system to resist a significantly larger lateral load than falsework. The lateral load that the temporary support system is required to resist is typically found in the contract documents. The minimum lateral load for temporary supports is typically 10 percent of the supported dead load, as specified in *Contract Specifications* Section 48-3.02B, *Temporary Structures – Temporary Supports – Materials – Design Criteria*.

An often overlooked requirement found in the *Contract Specifications* is for the temporary support to resist lateral loads in any direction. The temporary support system must resist the lateral loads shown. Transferring lateral loads through the existing structure is only allowed when specifically permitted by the *Contract Specifications*.

Temporary supports are required to be mechanically attached to the supported structure and its foundation. The mechanical connection must accommodate movement resulting from vertical adjustments made to the temporary supports while providing restraint to lateral movement. Removing the connection for adjustment is not allowed. See Figure 9-4 for an example of a mechanical connection that allows adjustment vertically.

Lateral forces induced on temporary supports may be resisted by the partially constructed, demolished, or augmented structure if it can be shown through analysis and calculations that such components have adequate stability and capacity.

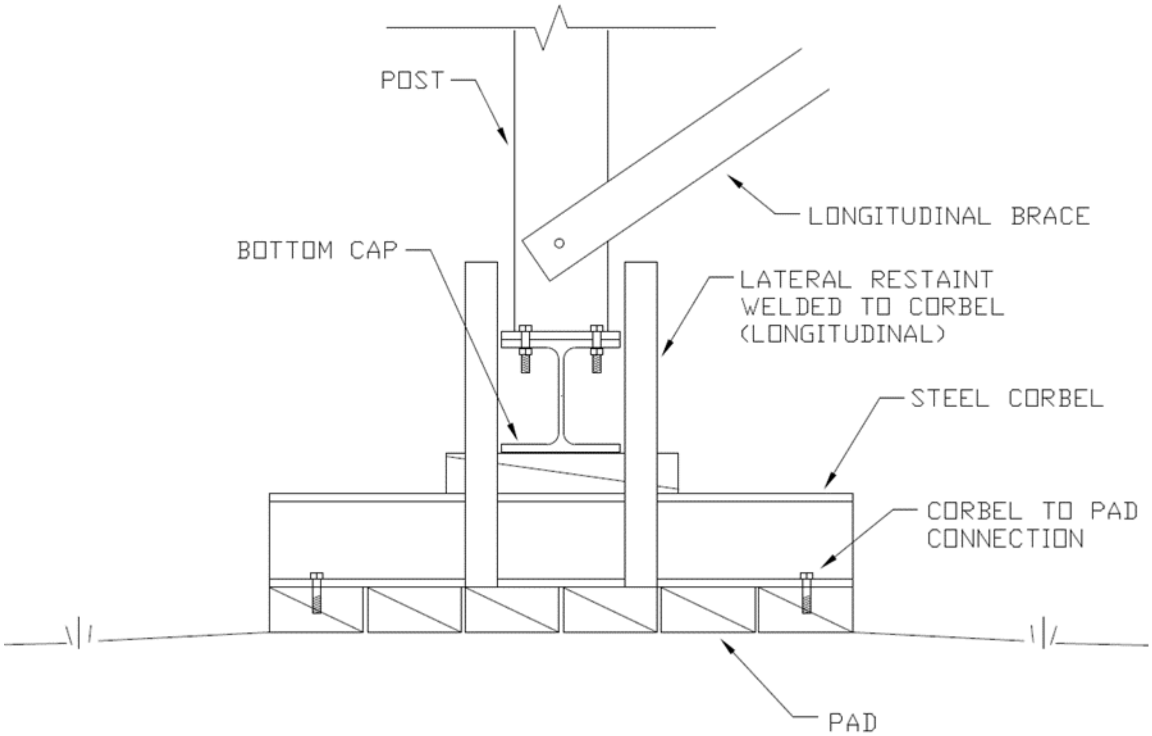

Figure 9-4. Typical Mechanical Connection Allowing Adjustment (vertical)

9-6 Jacking System Design

The design of the jacking system must support the dead load and lateral design forces plus additional loads from jacking equipment and activities. It is common for the jacking load to be greater than the supported dead load to initiate the initial movement of the structure.

During jacking operations, a redundant system of support must be provided, as illustrated in Figure 9-5. Lock rings on jacks are not considered a redundant system of support. Typical redundant systems consist of stacks of metal plates adjacent to the jacks. Alternative plate material is acceptable provided it can support the load without being overstressed or resulting in excessive deflection.

The quality assurance requirements of the *Contract Specifications* Section 48-5, *Temporary Structures – Jacking,* require jacks to be calibrated by an authorized laboratory within 6 months of use or after repair. The list of authorized laboratories can be verified with the Materials Engineering and Testing Services (METS) Representative.

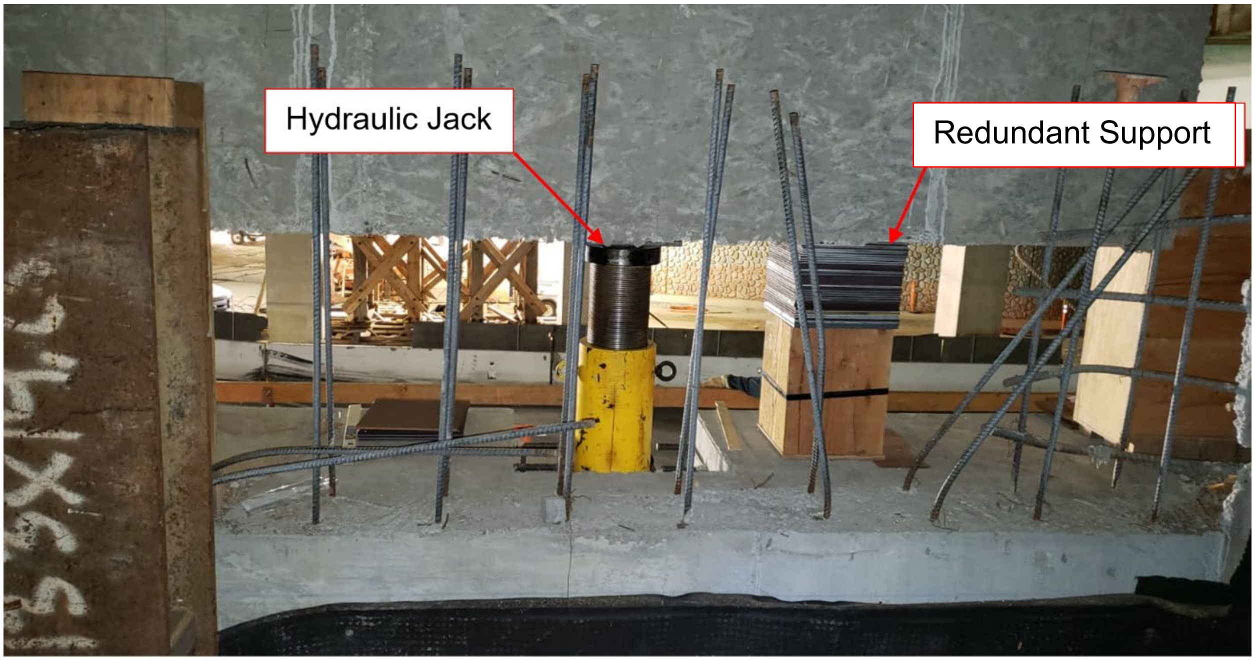

Figure 9-5. Typical Jacking System with Redundant System of Support

Appendix A Example 1: Wind Loads

This appendix covers the topic of wind loads. It uses three different methods to calculate wind loads, including Caltrans standard specifications, ASCE 7-16, and an AASHTO method. The results of the methods are then tabulated and compared.

Given Information

- Containment structure 42 feet high and 15 feet wide starting at grade
- Supported at top and base
- Located adjacent to traffic
- Ground elevation = 1000 ft

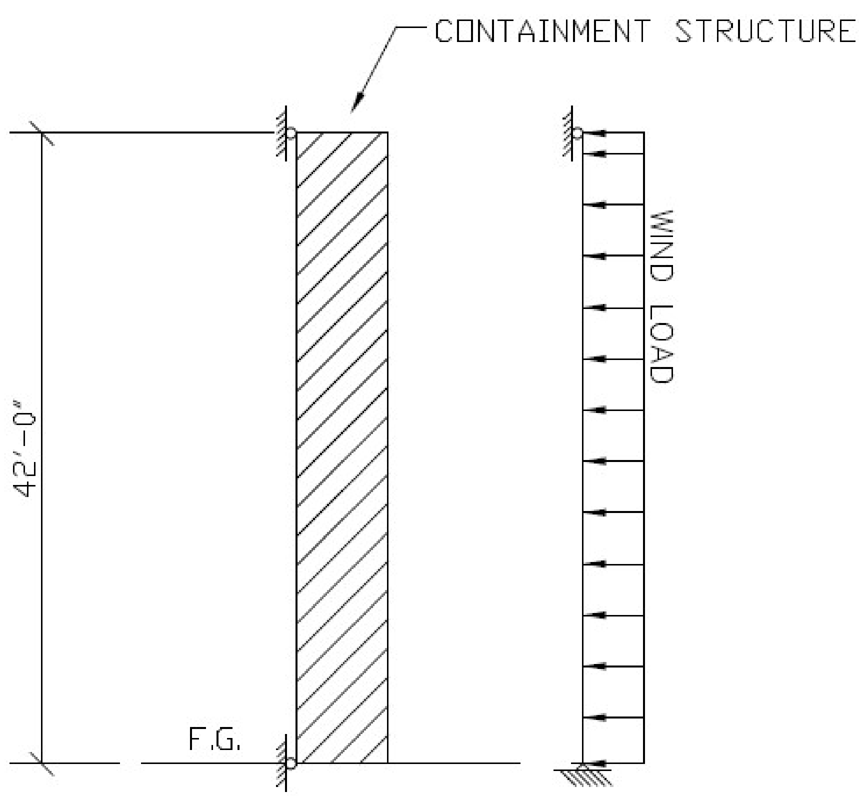

Figure A-1-1. Containment Structure Section

Required

1. Determine wind pressure using the wind pressure tables in *Contract Specifications* Section 48-2.02B(2), *Temporary Structures – Falsework – Materials – Design Criteria – Loads.*
2. Determine the wind pressure using ASCE 7-16.
3. Determine the wind pressure using AASHTO *Guide Design Specification for Bridge Temporary Works* (GSBTW).
4. Compare the results of the three methods above.
5. Determine the total force for each method above at top and bottom support.

1 - Wind Pressure Table Method (Contract Spec. 48-2)

Height zone, H (feet above ground)	Wind pressure value	
	Shores or columns adjacent to traffic (psf)	At other locations (psf)
H≤30	20	15
30<H≤50	25	20
50<H≤100	30	25
H>100	35	30

Figure A-1-2. Wind Pressure Table from Section 48-2.02B(2)

Height zone 0 to 30 feet pressure = 20 psf(15')(30') = 9000 lbs
Height zone 30 to 42 feet pressure = 25 psf(15')(12') = 4500 lbs
Total force = 13500 lbs

Total force on top support ($\mathbf{F_t}$):

$$F_t = \frac{9000(15) + 4500(36)}{42} = 7071 \text{ lbs} \quad \text{(A-1-1)}$$

Total force on bottom support ($\mathbf{F_b}$):

$$F_b = 13500 - 7071 = 6429 \text{ lbs} \quad \text{(A-1-2)}$$

2 - ASCE 7-16 Method

Determine the basic wind pressure using the procedures in ASCE 7-16, Chapter 26, *Wind Loads: General Requirements* and Chapter 29, *Wind Loads on Building Appurtenances and Other Structures* (this and other resources may be obtained from the Transportation Library[1]).

Given

Basic wind speed from Figure A-1-3 below (assume Risk Category II)

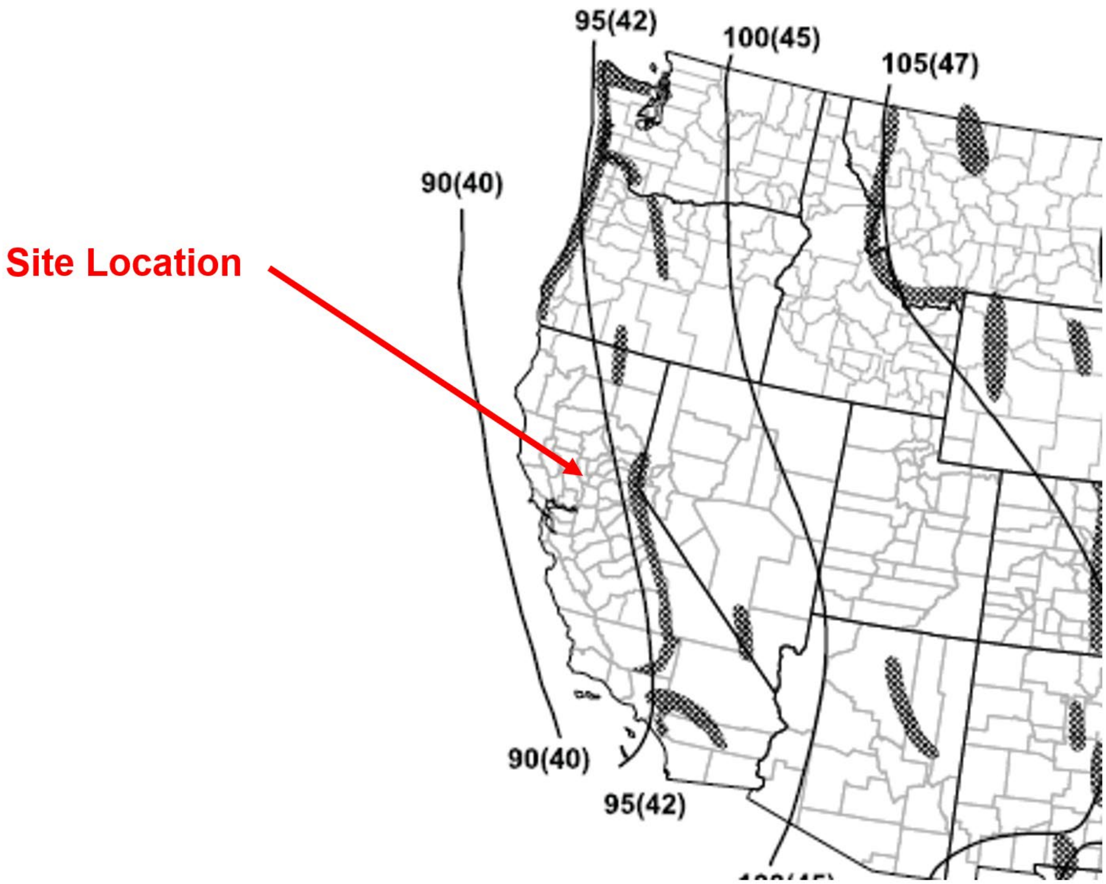

Figure A-1-3. Basic wind speeds for Risk Category II Buildings and Other Structures (ASCE 7-16 Figure 26.5-1B)

Basic wind speed (V) = 93 mph (interpolated based on site location)

The surface roughness and exposure are site specific

[1] Caltrans internal use only

Surface Roughness category = B (ASCE 26.7.2)

Exposure category = B (ASCE 26.7.3)

Topographic effects over hills, ridges, escarpments are determined using the topographic factor (**K_{zt}**) found in ASCE 26.8.2. For this example, there are no topographic effects which results in **K_{zt}** =1.0.

The ground level for this example is 1000 feet. From ASCE 7-16 Table 26.9-1, the ground elevation factor (**K_e**) = 0.96

Velocity pressure exposure coefficient from ASCE 7-16 Table 26.10-1 (shown below):

K_z for height above ground (**z**) 42 feet = 0.77 interpolated value (ASCE Table 26.10-1)

The wind directionality factor (**K_d**) = 0.85 (ASCE Table 26.6-1)

The velocity pressure (ASCE equation 26.10-1) at height **z** is shown below:

$$q_z = 0.00256\,(K_z)(K_{zt})(K_d)(K_e)(V^2) \tag{A-1-3}$$

Therefore, **q_z** for height 42 feet is:

$$q_z = 0.00256(0.77)(1.0)(0.85)(0.96)(93)^2 = 13.9\text{ psf} \tag{A-1-4}$$

Using ASCE 7-16 Chapter 29, *Wind Loads on Building Appurtenances and Other Structures*- from section 29.3 *Design Wind Loads: Solid Freestanding Walls and Solid Signs*:

Design wind force (**F**)	=	$q_h G C_f A_s$ (ASCE Eq 29.3-1)
q_h = q_z	=	13.9 psf
Gust factor (**G**)	=	0.85 (ASCE 26.11.1)
s/h = 42/42	=	1 (ASCE Figure 29.3-1)
B/s = 15/42	=	0.36 (ASCE Figure 29.3-1)
Force coefficient (**C_f**)	=	1.60 (interpolated from ASCE Figure 29.3-1)
Projected area (**As**)	=	630 sqft

$$F = 13.9(0.85)(1.60)(630) = 11910 \text{ lbs} \tag{A-1-5}$$

Force applied at $\frac{42}{2} + 0.05(42) = 23.10 \text{ ft}$ (common practice to apply at 55 % height) (A-1-6)

Design forces calculated above are strength design. For allowable stress design the design force is multiplied by 0.6 per ASCE 2.4 *Load Combinations for Allowable Stress Design.* Loads specified by the designer are service loads and are not reduced by the 0.6 factor. A wind load specified by the designer for removal of the containment system is an example of a service load that is not reduced.
Total force on top (**F_t**) support:

$$F_t = 0.6\left(\frac{23.10}{42}\right)(11910) = 3930 \text{ lbs} \tag{A-1-7}$$

Total force on bottom (**F_b**) support:

$$F_b = 0.6\,(11910) - 3930 = 3216 \text{ lbs} \tag{A-1-8}$$

Verify conformance with minimum loading (16 psf) of ASCE 29.7:

$$F_{min} = \frac{0.6(16)(15)(42)}{2} = 3024 \text{ lbs} < 3216 \text{ lbs} \tag{A-1-9}$$

3 - AASHTO GSBTW Method

Determine the wind pressure using AASHTO *Guide Design Specification for Bridge Temporary Works* (GSBTW)

Given

Basic wind speed from AASHTO Bridge Design Specification (BDS) is shown in Figure A-1-4 below:

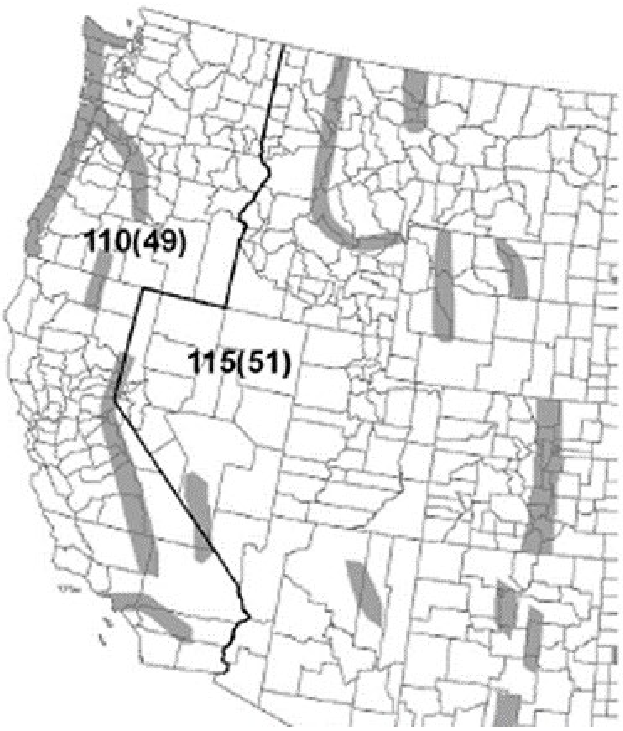

Figure A-1-4. Basic wind speed from AASHTO Bridge Design Specification (Figure 3.8.1.1.2-1)

Figure 3.8.1.1.2-1 is based on ASCE 7-10 with values corresponding to 3-second gust wind speeds at 33 feet above ground for Exposure C. Future revisions of the BDS will use wind maps found in the latest version of ASCE 7.

Basic wind speed (**V**) = 110 mph

Wind exposure category = **B** (used in previous example)

Pressure exposure coefficient at **h**= 42 feet ($\mathbf{K_z}$) = 0.76 (AASHTO BDS Table C3.8.1.2.1-1)

Gust coefficient (**G**) = 0.85 (GSBTW 2.3.5.2.3b)

Drag coefficient ($\mathbf{C_D}$) for solid surface = 2.0 (GSBTW Table 2.3.5.2.3b-2)

Wind directionality factor ($\mathbf{K_d}$) = 0.95 (GSBTW 2.3.5.2.3b)

Design wind pressure $\mathbf{P_z} = 0.00256(K_z)(G)(C_d)(K_d)(V^2)$

$\mathbf{P_z}$ for height 42 feet = $0.00256(0.76)(0.85)(2.0)(0.95)(110^2) = 38.02$ psf

Increase $\mathbf{P_z}$ by 5 psf for members over or adjacent to traffic per GSBTW 2.3.5.2.3b

$\mathbf{P_z}$ = 43.02 psf

F= $\mathbf{P_z}$(A)

A = Area projected on vertical plane

Design forces calculated above are strength design. For allowable stress design the design force is multiplied by 0.6 per GSBTW Table 2.3.2.2-1 *Load Combinations and Load Factors.*

Total force **F**:

$$\mathrm{F} = 0.6\,(43.02)(15)(42) = 16262\ \mathrm{lbs} \tag{A-1-10}$$

Total force on top ($\mathbf{F_t}$) and bottom ($\mathbf{F_b}$) support Case 1 (GSBTW 2.3.5.2.3d)

$$\mathrm{F_t} = \mathrm{F_b} = \frac{16262}{2} = 8131\ \mathrm{lbs} \quad \text{(force applied at centroid)} \tag{A-1-11}$$

Total force on top ($\mathbf{F_t}$) support Case 2 (GSBTW 2.3.5.2.3d)

$$\mathrm{F_t} = \left(\frac{23.10}{42}\right)(16262) = 8944\ \mathrm{lbs} \quad \text{(Total Force applied at 0.55 height of gross area)} \tag{A-1-12}$$

$$\mathrm{F_b} = 16262 - 8944 = 7318\ \mathrm{lbs} \quad \text{(Total Force applied at 0.55 height of gross area)} \tag{A-1-13}$$

4 - Comparison of Methods

Table A-1-1. Comparison of Methods

Design Method	Total Force (lbs)	Force at Top (lbs)	Force at Bottom (lbs)
CT Pressure Table	13500	7071	6429
ASCE 7-16	7146	3930	3216
AASHTO GSBTW	16262	8944	7318

The wind pressure tables in *Contract Specifications* Section 48-2, *Loads*, are an acceptable method to determine forces on other temporary structures. If the temporary structure design is capable of resisting the wind pressure loads in Section 48-2, no further analysis is needed.

If it is determined the temporary structure cannot resist the loading using the wind pressure tables in Section 48-2, then further analysis using the wind pressure method the temporary structure designer used is required to determine if the structure can resist the calculated wind loads.

Appendix A Example 2: Column Guying

This appendix covers the topic of column guying. The wind load on a typical rebar cage/ column form is calculated, and the guying system checked for adequacy.

Given Information

- 3'-0" diameter column
- Column reinforcement cover 3 inches
- Pile cutoff 3'-6" below finish grade
- Assume column form is pinned at top of pile
- Deadman dimensions are 4 ft x 4 ft x 4 ft
- For cable attachments to deadman see Figure A-2-5

Required

1. Review guying submittal and verify conformance with *Contract Specifications* Section 52-1.01C(2).
2. Verify deadman capacity to resist loads.
3. Verify cable loads are less than allowable.
4. Check cable connection at deadman.

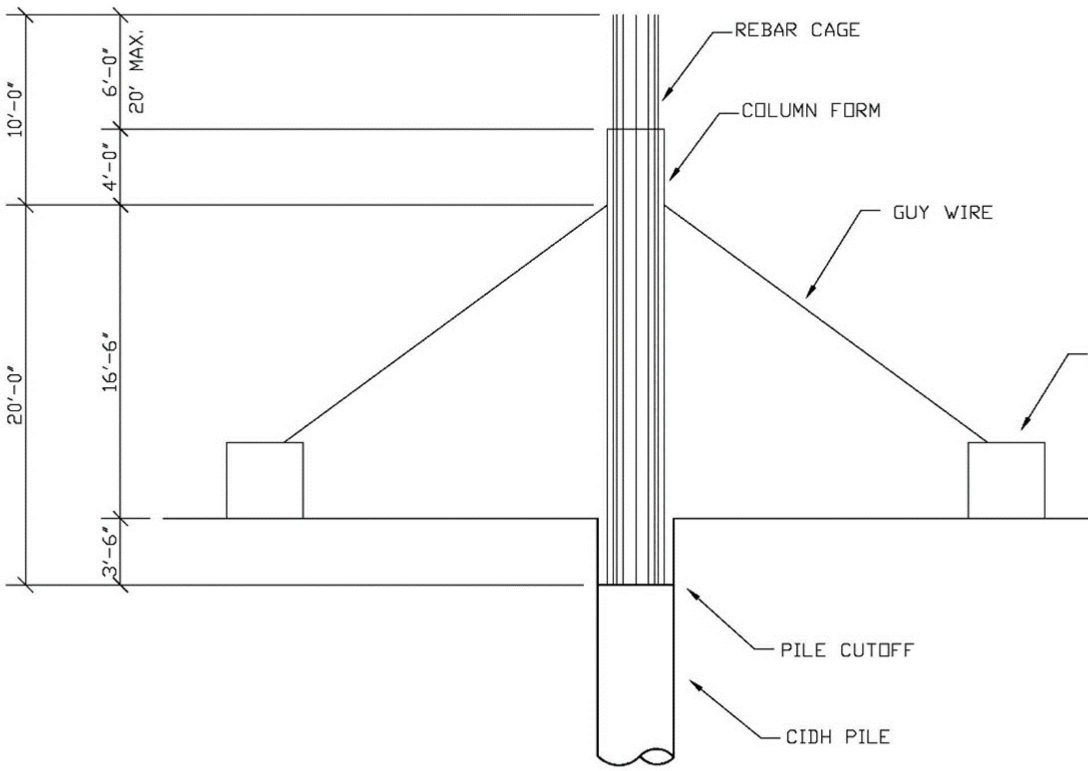

Figure A-2-1. Elevation Column Support

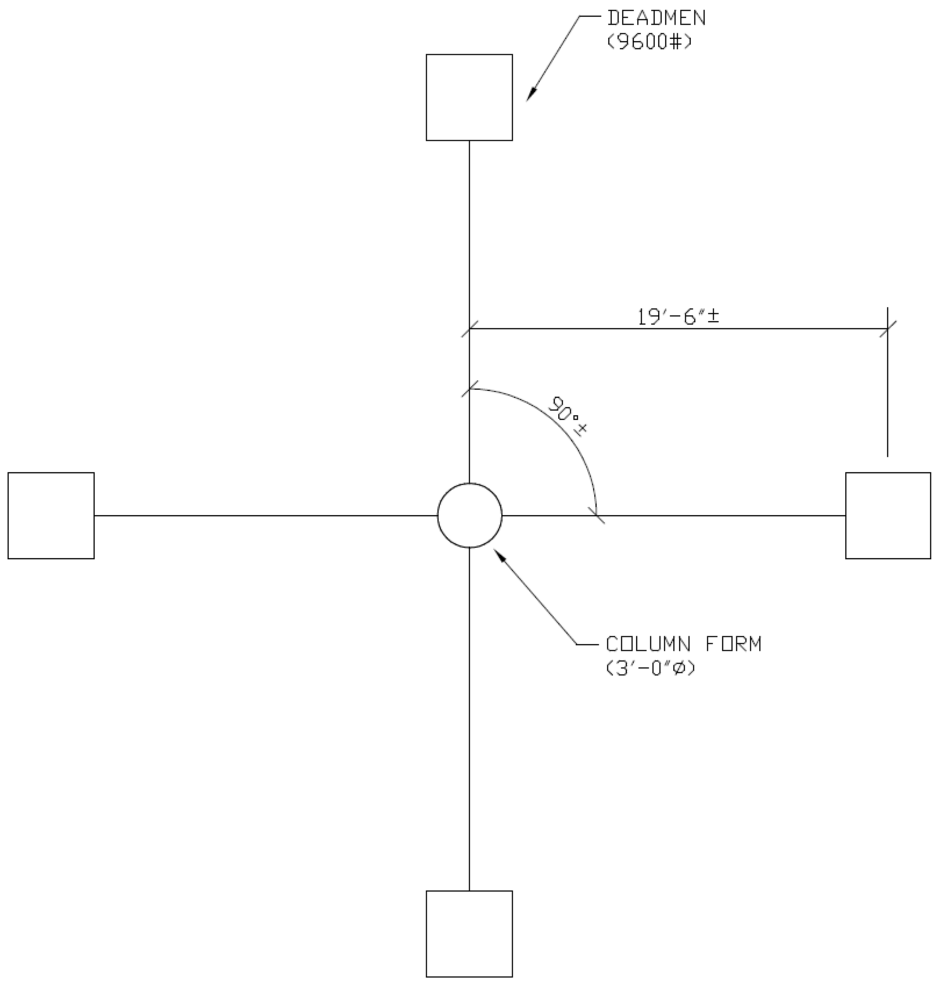

Figure A-2-2. Plan View Column Support

Loading

Wind pressures from *Contract Specifications* Section 52-1.01C(2), *Reinforcement — Shop Drawings*, taken from table below for the associated height zone:

Wind Pressure

Height zone, H (feet above ground)	Wind pressure value (psf)
0 ≤ 30	20
30 < H ≤ 50	25
50 < H ≤ 100	30
H > 100	35

Figure A-2-3. Wind Pressure Values

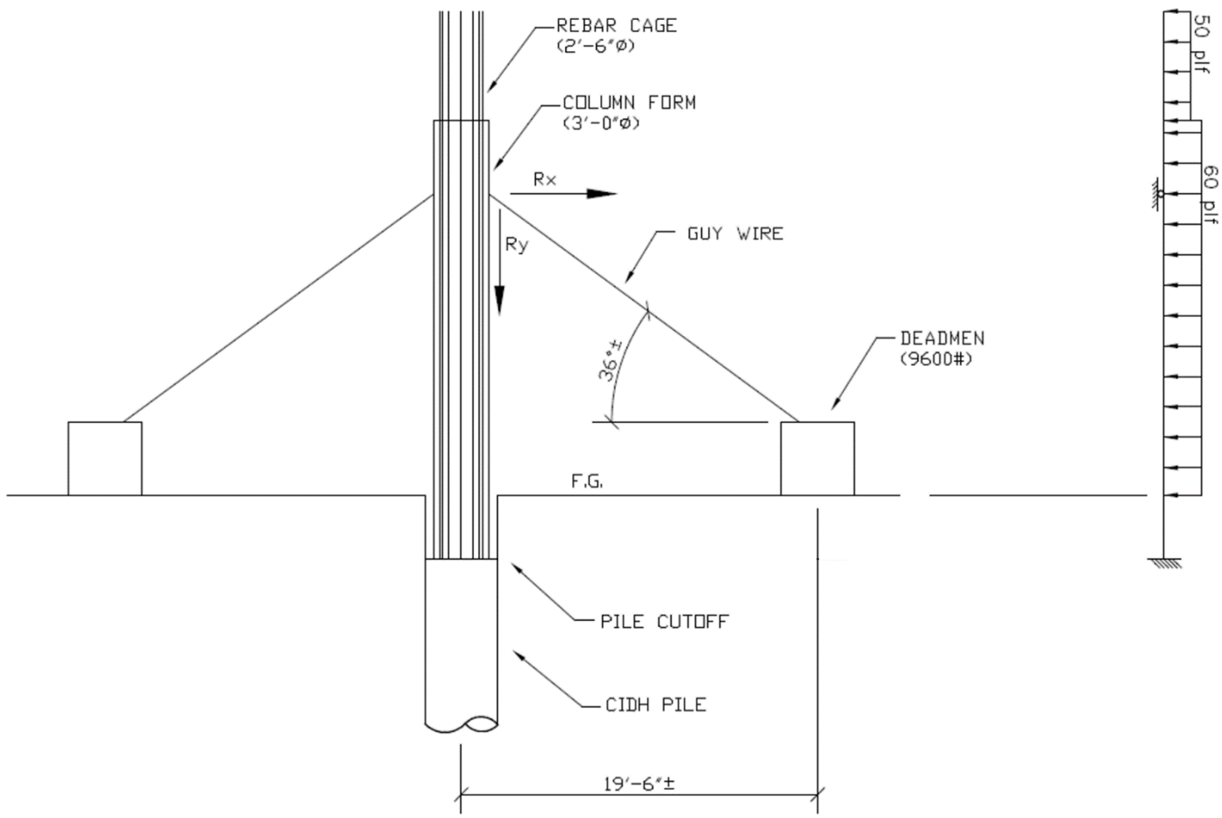

Figure A-2-4. Guying System Loading

Cable forces

Determine cable support reaction using beam theory by summing moments about the base. Conservatively assume the column is pinned at the base to simplify the calculations. A more detailed analysis using beam software could be used.

Cable reaction (**R_x**):

$$\mathrm{R_x} = \frac{(60\ \mathrm{plf})(20.5\ \mathrm{ft})\left(\frac{20.5\ \mathrm{ft}}{2} + 3.5\ \mathrm{ft}\right) + (50\ \mathrm{plf})(6\ \mathrm{ft})\left(\frac{6\ \mathrm{ft}}{2} + 24\ \mathrm{ft}\right)}{20\ \mathrm{ft}} \approx\ 1250\ \mathrm{lbs} \quad \text{(A-2-1)}$$

Cable reaction (**R_y**):

$$\mathrm{R_y} = 1250\ \mathrm{lbs}\ (\tan 36°)\ =\ 908\ \mathrm{lbs} \quad \text{(A-2-2)}$$

Resultant (cable tension):

$$\sqrt{R_x{}^2 + R_y{}^2} = \sqrt{1250^2 + 908^2} = 1545 \text{ lbs} \qquad \text{(A-2-3)}$$

<u>Support Cables</u>

Cable reaction = 1545 lbs (previously calculated)

For 1/2" 6x19 IWRC cable:

Minimum breaking strength (**MBF**)	=	13.30 tons = 26,600 lbs (Contractor to provide manufacturer's data in submittal)
Factor of safety (**FS**)	=	2 (guying not required to adhere to FS found in the *Falsework Manual*)

Connection: 3 wire rope clips (Crosby)

Connection efficiency (**CE**)	=	80 % (*Falsework Manual,* Table 5-2, Wire Rope Connections)

Allowable cable load:

$$\frac{\text{MBF(CE)}}{\text{FS}} = \frac{26{,}600(0.80)}{2} = 10640 \text{ lbs} > 1545 \text{ lbs} \quad \Longleftarrow \quad \text{OK} \qquad \text{(A-2-4)}$$

<u>Cable Attachment at Deadmen</u>

Cable tension	=	1,545 lbs (previously calculated)
Angle to horizontal	=	36°
$\mathbf{R_x}$	=	1,250 lbs
$\mathbf{R_y}$	=	908 lbs

Anchor spacing and edge distance as illustrated below:

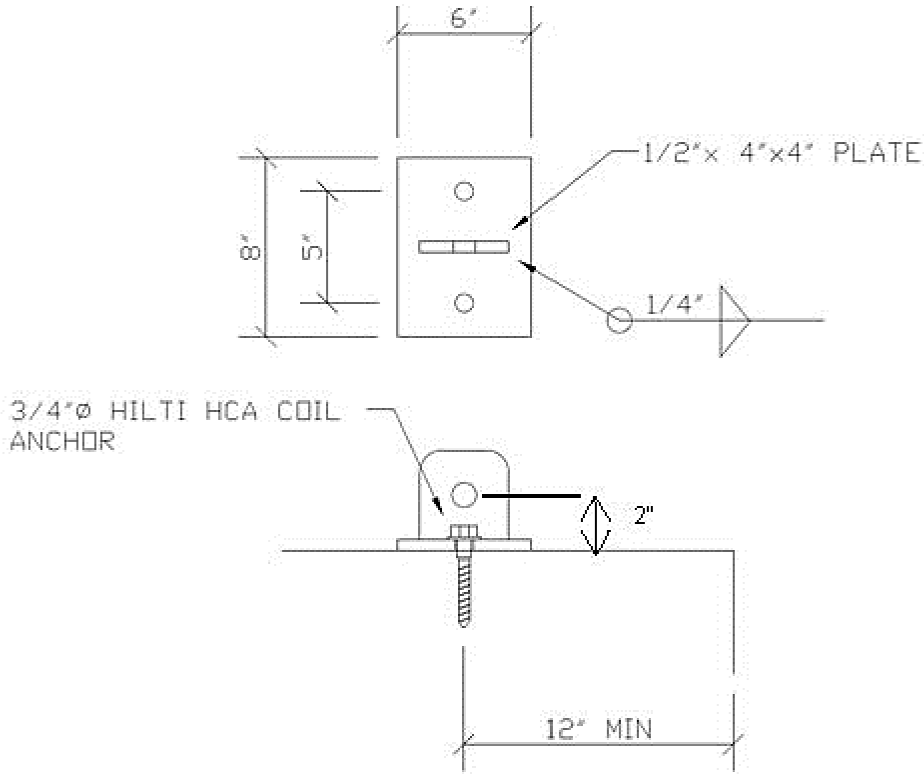

Figure A-2-5. Cable Attachment at Deadmen

Anchor Analysis

For two ¾" Hilti HCA coil anchors:

Nominal embedment	=	4.5" min
Edge distance	=	12" min
Spacing	=	5"
Assume $\mathbf{f'_c}$	=	3000 psi
Vertical load $\mathbf{R_y}$	=	908 lbs (previously calculated)
Horizontal load $\mathbf{R_x}$	=	1250 lbs (previously calculated)

Anchor Manufacturer's Instructions

MATERIAL SPECIFICATIONS

1/4-in. HCA manufactured from case hardened AISI 1038 carbon steel with a minimum tensile strength of 100 ksi (690 MPa).
3/8-, 1/2-, 5/8- and 3/4-in. HCA meet the chemical requirements of AISI 1035 carbon steel and are heat treated for a minimum tensile strength of 120 ksi (830 MPa).
Coil is manufactured from carbon steel.
Anchor and coil are zinc plated in accordance with ASTM B633, SC 1.

Figure 1 - HCA specifications

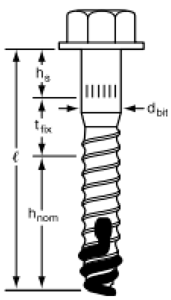

Table 1 - Hilti HCA Coil Anchor specifications

Setting information		Symbol	Units	Nominal anchor diameter				
				1/4	3/8	1/2	5/8	3/4
Nominal bit diameter		d_o	in.	1/4	3/8	1/2	5/8	3/4
Embedment mark[1]		h_s	in.	3/8	5/8	5/8	3/4	1
Anchor Length	min.	ℓ	in.	1-3/4	2-1/4	3	3-1/2	4-1/2
	max.	ℓ	in.	3-1/2	5	7	8	10
Fixture hole diameter		d_h	in.	5/16	7/16	9/16	11/16	13/16
Installation torque		T_{inst}	ft-lb	10	40	80	130	180
Minimum base material thickness		h	in.	the greater of 3 or 1.3 times h_{nom}				

1 Maximum fixture thickness $t = \ell - (h_{nom} + h_s)$

Combined shear and tension loading

$$\left(\frac{N_d}{N_{rec}}\right) + \left(\frac{V_d}{V_{rec}}\right) \leq 1.0$$

Figure A-2-6. Anchor Manufacturer's Instructions

HILTI

Table 2 - Hilti HCA allowable concrete and steel capacity (lb)[1]

Nominal anchor diameter in.	Nominal embedment in.	f'_c = 2,000 psi		f'_c = 4,000 psi		f'_c = 6,000 psi		Allowable steel strength[2]	
		Tension[3]	Shear	Tension[3]	Shear	Tension[3]	Shear	Tension	Shear
1/4	3/4	230	230	325	330	400	400	1,620	835
	1	355	380	500	535	615	655		
3/8	1-1/2	650	850	920	1,205	990	1,475	4,375	2,255
	2	1,005	1,390	1,420	1,965	1,740	2,410		
1/2	2	1,005	1,515	1,420	2,145	1,740	2,625	7,775	4,005
	3	1,845	3,020	2,605	4,270	3,190	5,230		
5/8	2-3/8	1,300	2,175	1,835	3,075	2,250	3,765	12,150	6,260
	3-7/8	2,705	5,000	3,825	7,070	4,685	8,660		
3/4	3-1/4	2,080	3,915	2,940	5,540	3,600	6,780	17,495	9,010
	4-1/2	3,385	6,810	4,790	9,630	5,865	11,705		

1 Allowable concrete capacities based on a safety factor of 4.
2 Steel strength calculated using 0.33 f_{uta} $A_{nominal}$ for tension and 0.17 f_{uta} $A_{nominal}$ for shear.
3 Reduce tension capacity by 20% for HCA Hex Head Bolts that are reused. Coils may not be reused.

Figure A-2-7. Hilti HCA Allowable Concrete and Steel Capacity

Table 4 - Hilti HCA edge distance and anchor spacing guidelines[1,2]

Load Direction		Critical	Minimum	Influence factor[3]
Spacing	Tension	3.0 h_{nom}	1.0 h_{nom}	f_{AN} = 0.70
Spacing	Shear	2.0 h_{nom}	1.0 h_{nom}	f_{AV} = 0.70
Edge distance	Tension	1.5 h_{nom}	0.8 h_{nom}	f_{RN} = 0.75
Edge distance	Shear ⊥ toward edge[4]	2.5 h_{nom}	1.0 h_{nom}	f_{RV1} = 0.25
Edge distance	Shear ll or ⊥ away from edge[4]	2.5 h_{nom}	1.0 h_{nom}	f_{RV2} = 0.50

1 For edge and spacing distances between critical and minimum spacing/edge distances, use linear interpolation.

2 Influence factors are cumulative.

3 Influence factor at minimum spacing/edge distance. Influence factor at critical equals 1.0.

4 For shear loads in between perpendicular toward edge and parallel with edge, use the following equation, f_{RV3} = 0.25 / (cos ß + 0.5 sin ß) for 55° ≤ ß < 90°. For 0° ≤ ß < 55°, use influence factor for shear perpendicular toward edge. See Figure 2.

Figure A-2-8. Hilti HCA Edge Distance and Anchor Spacing Guidelines

Tension

Maximum anchor plate thickness (t) = l – (H_{nom}+h_s) = 6" – (4.5"+1") = 0.5" max (Figure A-2-6)

Note: Verify anchor plate thickness less than or equal to maximum ½" in field; if larger than ½", a longer anchor is required.

Minimum spacing shear and tension = 1.0(h_{nom}) = 1.0(4.5") = 4.5"

Critical spacing tension = 3(h_{nom}) = 3(4.5") = 13.5"> 5" (Reduction required)

Critical edge Dist. tension = 1.5(h_{nom}) = 1.5(4.5") = 6.75" < 12" (No reduction for edge Dist.)

Critical spacing shear = 2(h_{nom}) = 2(4.5") = 9" > 5" (Reduction required)

Critical edge Dist. shear = 2.5(h_{nom}) = 2.5(4.5") = 11.25" < 12" (No reduction for edge Dist.)

Determine spacing influence factors using linear interpolation between critical and minimum distances.

Spacing influence factor for tension:

$$0.7 + 0.3\left(\frac{5 - 4.5}{13.5 - 4.5}\right) = 0.717 \qquad \text{(A-2-5)}$$

Spacing influence factor for shear:

$$0.7 + 0.3\left(\frac{5 - 4.5}{9 - 4.5}\right) = 0.733 \tag{A-2-6}$$

Allowable tension load:

$$2\left(\frac{3385 + 4790}{2}\right)(0.717) = 5{,}861 \text{ lbs} > 908 \text{ lbs} \quad \Leftarrow \quad \text{OK} \tag{A-2-7}$$

Allowable shear load:

$$2\left(\frac{6810 + 9630}{2}\right)(0.733) = 12{,}050 \text{ lbs} > 1250 \text{ lbs} \quad \Leftarrow \quad \text{OK} \tag{A-2-8}$$

Combined loading:

$$\left(\frac{908}{5861}\right) + \left(\frac{1250}{12050}\right) = 0.26 < 1 \quad \Leftarrow \quad \text{OK} \tag{A-2-9}$$

Cable Attachment Plate

Load to welds:

R_x = 1,250 lbs (previously calculated)
R_y = 908 lbs (previously calculated)
R = 1,545 lbs

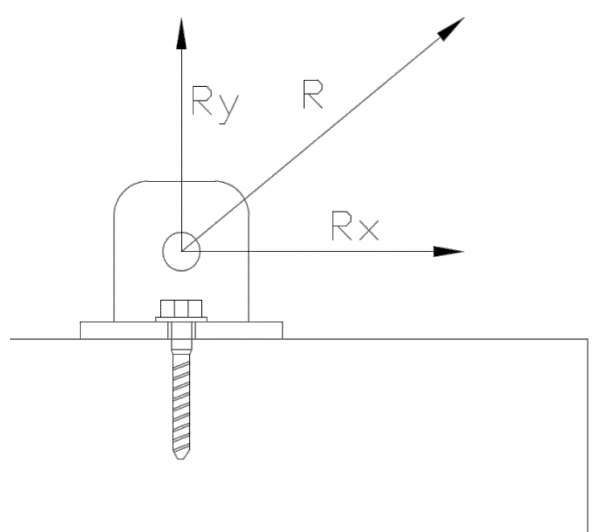

Figure A-2-9. Resultant Forces on Anchor Plate

Weld Capacity per AISC Chapter J:

For ¼" fillet welds:

$$A_w = 0.25"(0.707)(2)(4") = 1.414 \text{ in}^2 \tag{A-2-10}$$

$$M_w = R_x(2) = 1250(2) = 2500 \text{ in-lbs} \tag{A-2-11}$$

$$I_w = \frac{bd^3}{12} = \frac{2(0.25)(0.707)(4)^3}{12} = 1.885 \text{ in}^4 \tag{A-2-12}$$

Determine capacity of weld assuming a uniform shear distribution and combining the shear, bending, and axial stresses vectorially. This is conservative since the maximum bending stress and maximum shear stress occur at different locations.

Loading in the **y** direction:

$$f_t = \frac{P}{A} + \frac{Mc}{I} = \frac{908}{1.414} + \frac{2500(2)}{1.885} = 3295 \text{ psi} \tag{A-2-13}$$

Loading in the **x** direction:

$$f_v = \frac{V}{A} = \frac{1250}{1.414} = 884 \text{ psi} \tag{A-2-14}$$

Resultant load:

$$f_r = \sqrt{3295^2 + 884^2} = 3412 \text{ psi} \tag{A-2-15}$$

For ASD weld capacity:

$$R_n = \frac{F_w A_w}{\Omega} \tag{A-2-16}$$

$$\Omega = 2.00$$

$$F_w = 0.60F_{exx}(1 + 0.50(\sin^{1.5}\theta)) \quad \text{from AISC J2-5} \tag{A-2-17}$$

$$F_{wy} = 0.60(70 \text{ ksi})(1 + 0.50(\sin^{1.5}90)) = 63 \text{ ksi} = 63{,}000 \text{ psi}$$

$$\frac{F_{wy}}{\Omega} = \frac{63000}{2} = 31{,}500 \text{ psi}$$

$$F_{wx} = 0.60(70 \text{ ksi})(1 + 0.50(\sin^{1.5} 0)) = 42 \text{ ksi} = 42{,}000 \text{ psi}$$

$$\frac{F_{wx}}{\Omega} = \frac{42000}{2} = 21{,}000 \text{ psi}$$

$$F_{wr} = \sqrt{31500^2 + 21000^2} = 37{,}852 \text{ psi} > 3{,}412 \text{ psi} \quad \Longleftarrow \quad \text{OK} \qquad \text{(A-2-18)}$$

Deadmen:

Deadmen weight	=	9,600 lbs
Deadmen dimensions	=	4' x 4' x 4'
R_x	=	1,250 lbs (previously calculated)
R_y	=	908 lbs (previously calculated)
Soil type:	=	Gravel
Coefficient of friction	=	0.60 (dry soil)

Sliding resistance of Deadmen = 0.60(9600 – 908) = 5,215 lbs

$$\frac{1250}{5215} = 0.24 < 1 \quad \Longleftarrow \quad \text{OK} \qquad \text{(A-2-19)}$$

Overturning of Deadmen = 1250(4') + 908(1') = 5908 ft-lbs (moment taken about leading toe)

Deadmen resistance to overturning = 9600(2') = 19,200 ft-lbs

$$\frac{5908}{19200} = 0.31 < 1 \quad \Longleftarrow \quad \text{OK} \qquad \text{(A-2-20)}$$

Appendix A Example 3: Bridge Scaffolding

This appendix provides an example of bridge scaffolding which is suspended from a steel bridge. The calculations illustrated check the demand and adequacy of the scaffold support cables, as well as the available bridge load capacity.

Given Information

- Two span bridge with concrete deck and steel girders
- Column Steel girders are simply supported (span = 135')
- Flexible scaffold system suspended by cables from bridge structure
- Scaffold designed for 25 psf LL

Required

1. Review scaffold submittal for conformance with Standard Special Provisions Section 7-1.02K(6)(e), *Scaffolding*.
2. Verify global scaffold demand does not exceed available capacity.
3. Verify cable loads are less than allowable.
4. Check cable connection at abutment walls.

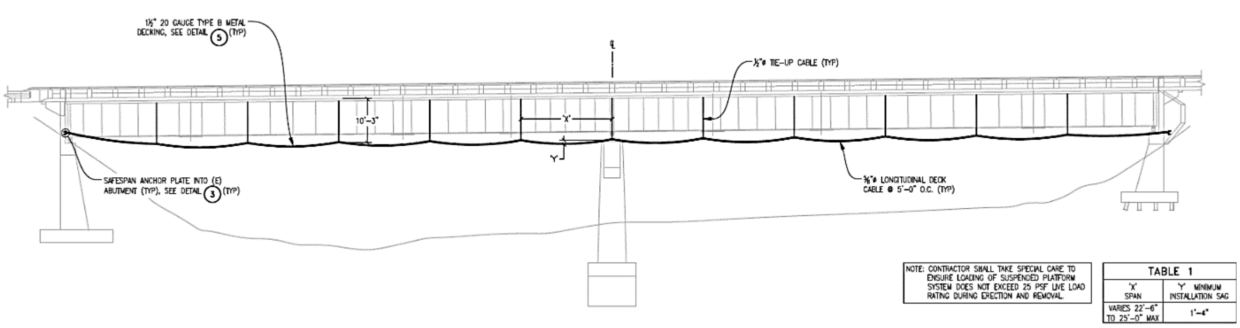

Figure A-3-1. Elevation

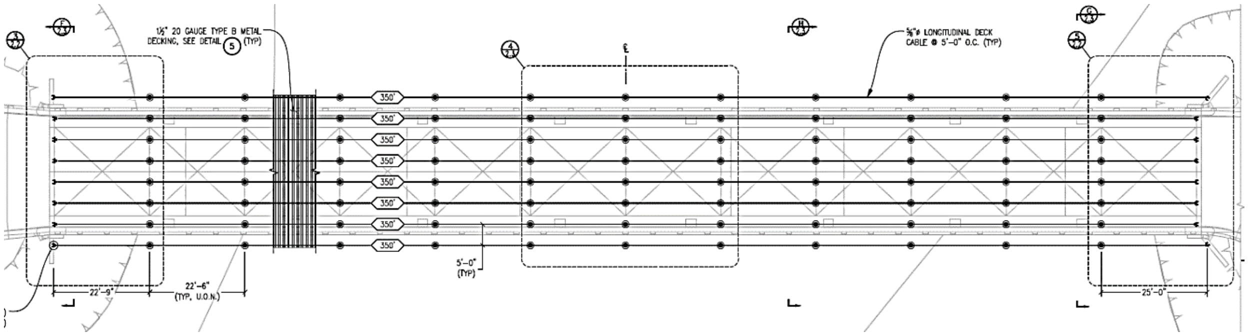

Figure A-3-2. Plan View Scaffold Layout

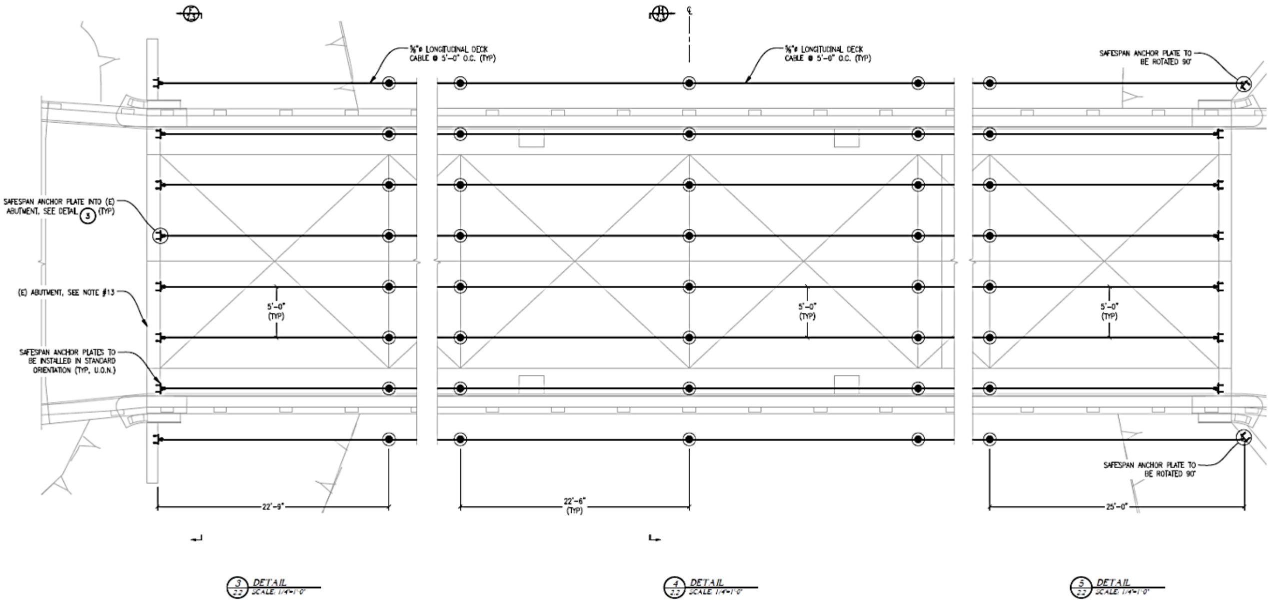

Figure A-3-3. Plan View Center and End Spans

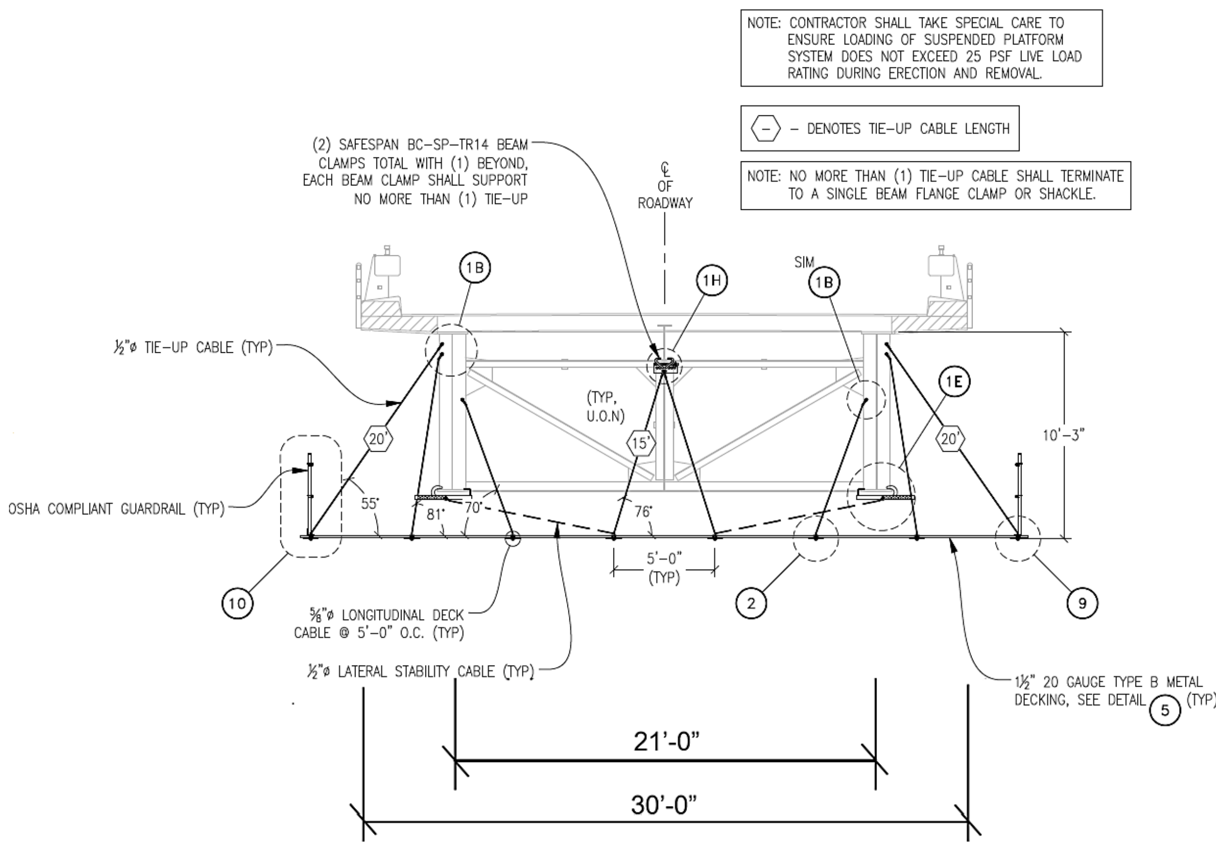

Figure A-3-4. Structure Section

Dead Load

Scaffold deck = 2.06 psf
Tie-up & Longitudinal Cable = 0.72 plf

Live Load

Review General Notes from shop drawings for proposed loading = 25 psf (proposed)

Wind Load

Wind loads calculated using documented method. For this example, the provisions of *AASHTO Guide Design Specifications for Bridge Temporary Works*, Second Edition (GSBTW) with 2020 Interim Revisions were used to calculate wind pressures to be applied to the scaffold structure.

Loads:

DL = 2.06 psf & 0.72 plf

LL = 25 psf

WL_{30} (wind at V= 30 mph sustained wind speed requires that workers vacate structure; provided by designer in submittal)

WL_{92} (wind at V= 92 mph site basic wind speed)

Load Combinations (ASD)

1. DL
2. DL+LL
3. DL+0.75LL+0.75WL_{30} (WL_{30} is service load)
4. DL+0.6WL_{92}
5. 0.6DL+0.6WL_{92} (for maximum uplift)

Note:
Wind at 30 mph (WL_{30}) is a service load and wind at 92 mph (WL_{92}) is a factored load. For ASD, the factored wind load is multiplied by 0.6.

Global Check of Existing Structure

Longitudinal cables spaced 5'-0" O.C. (see structure section above)

Overall scaffold width = 5'-0"(7) = 35'-0"

Bridge width = 30'-0"

Vertical supports spaced @ 22'-6" typical (see plan view scaffold layout above)

Available bridge uniform load = 65 psf (from contract specifications below):

2. Bluff Creek Bridge (br. no. 04-0063), Rube Creek Bridge (br. no. 04-0215) and Bluff Creek Bridge (br. no. 04-0225):

Bridge name/number	Available bridge load capacity (lb/sq ft)	Bridge width (ft)	HS-20 design live load		Permit design live load	
			Percentage of HS-20 loading (percent)	No. LL_{HS20} lanes	Permit vehicle	No. LL_{Permit} lanes
Bluff Creek Bridge (br. no. 04-0063)	65	30	100	2.38 for main girders	N/A	N/A

Figure A-3-5. Available Bridge Uniform Load for Bluff Creek Bridge

Girder spacing = 21'-0" (Figure A-3-4)

Available Bridge Load Capacity

Available capacity distributed loading = 30'(65 psf) = 1950 plf

Demand

Scaffold point dead load to bridge = 35'(22.5')(2.06psf)+ 8(22.5')(0.72plf) = 1752 lb

Scaffold point live load to bridge = 35'(22.5')(25psf) = 19688 lb

Total = 21440 lb

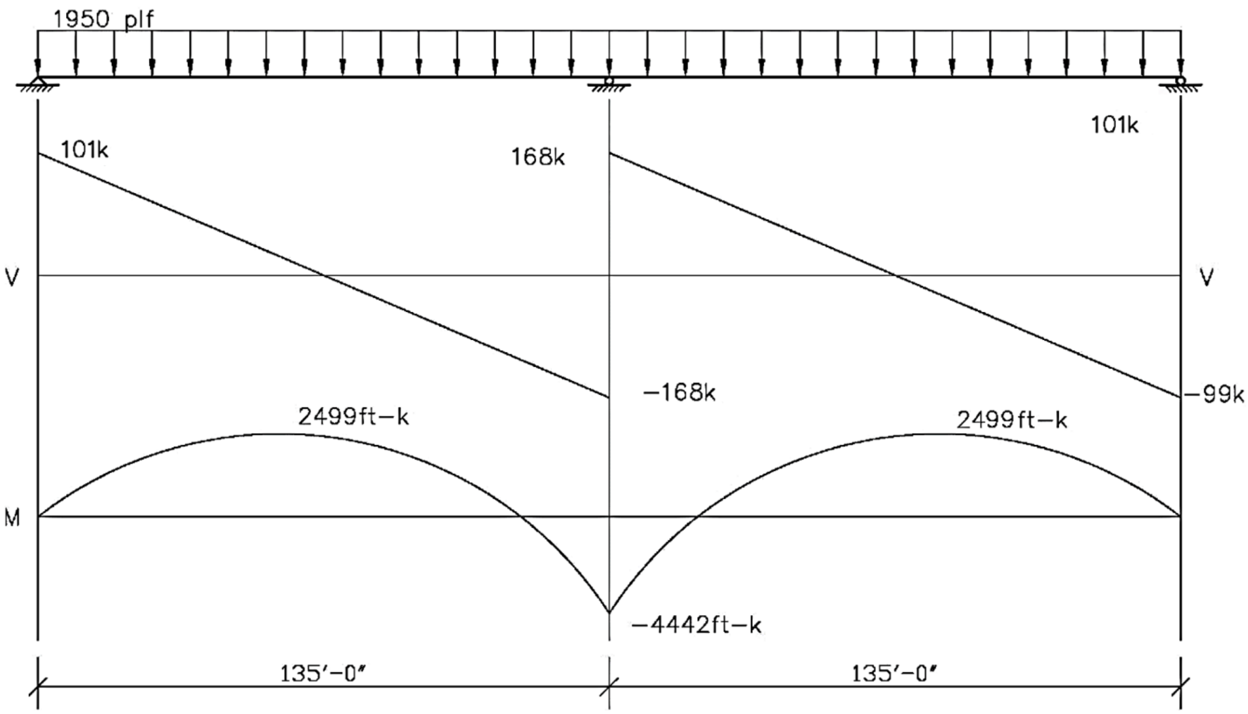

Figure A-3-6. Girder Capacity Shear and Moment Envelope

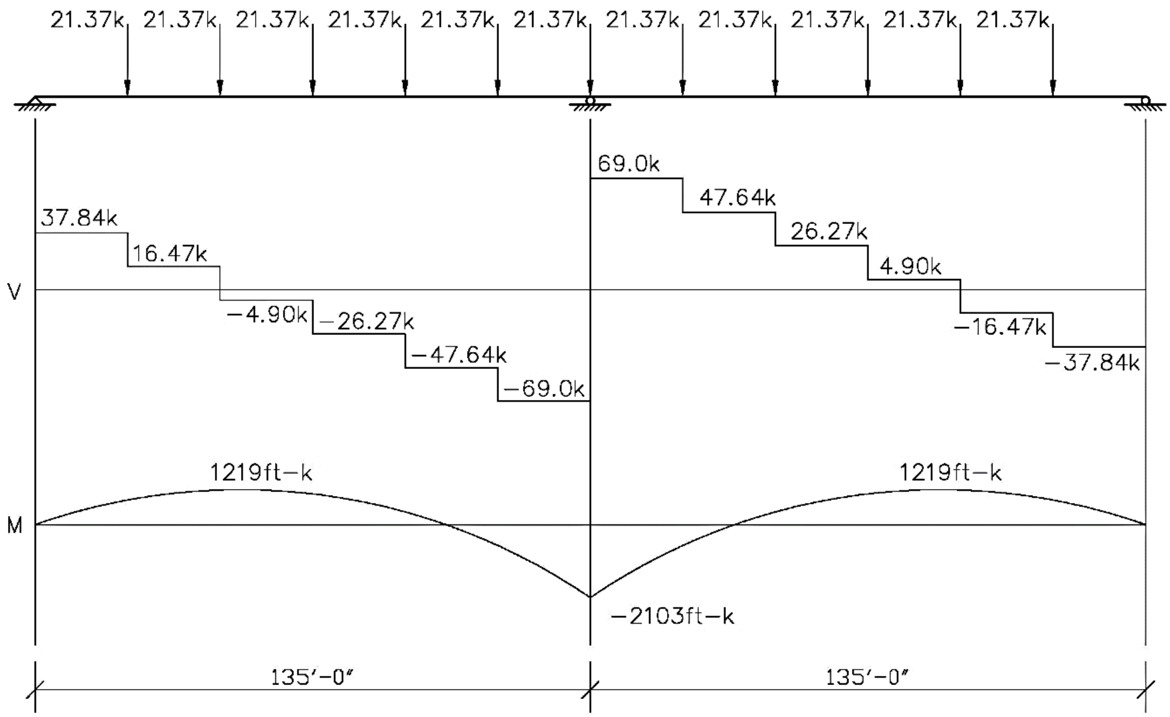

Figure A-3-7. Girder Demand Shear and Moment Envelope Full Live Load

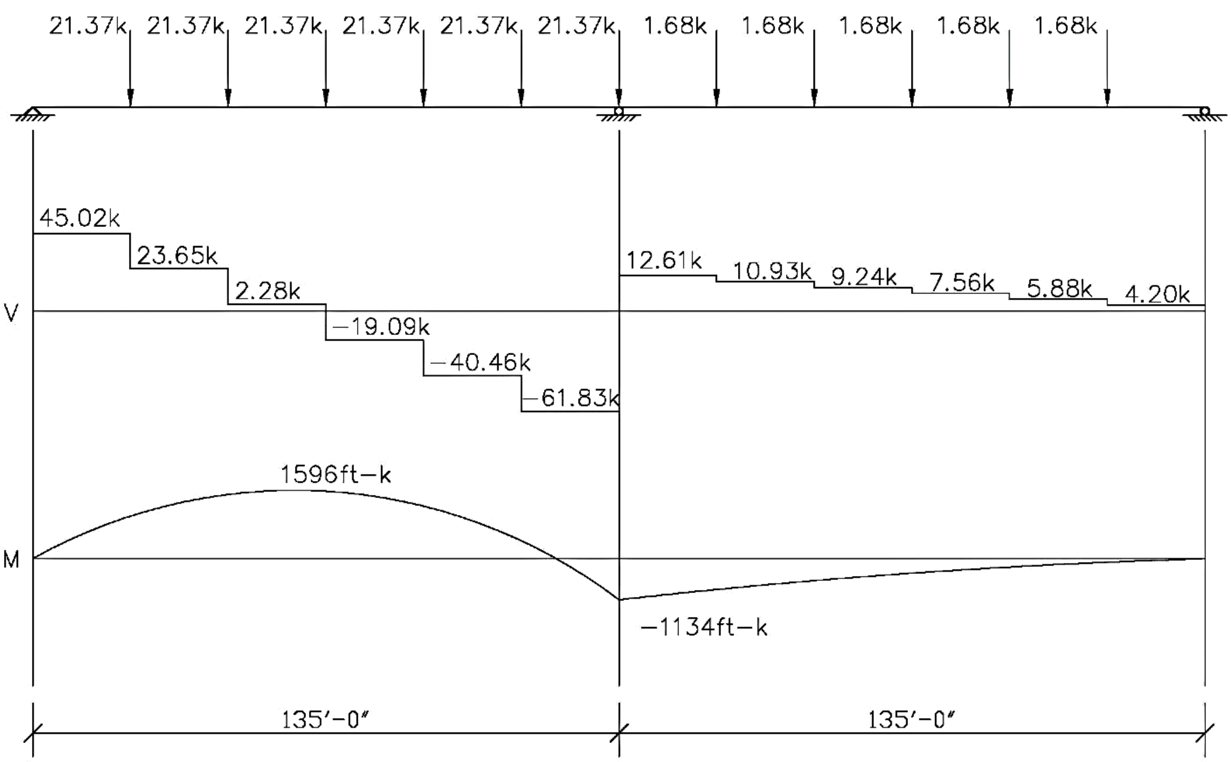

Figure A-3-8. Girder Demand Shear and Moment Envelope Alternate Span Live Load

Taking the highest demands from Figure A-3-7 and A-3-8:

Shear demand ends = 45.02 k < 101 k capacity ⟸ OK

Shear demand center = 69.00 k < 168 k capacity ⟸ OK

Moment demand Positive = 1596 ft-k < 2499 ft-k capacity ⟸ OK

Moment demand Negative = 2103 ft-k < 4442 ft-k capacity ⟸ OK

Longitudinal Cables (analyzed per typical cable)

Cable span	=	25'-0" max per shop drawings
Cable sag installed	=	1'-4" (75% of design sag per shop drawings)
Cable sag design	=	1'-9" (1.33x1'-4" = 1'-9" Includes construction and elastic stretch per shop drawings)
Cable load (**q**)	=	[2.06 psf +(25 psf)](5')+(0.72 plf) = 136 plf (interior condition) [2.06 psf +(25 psf)](2.5')+(0.72 plf) = 68 plf (25' ext. condition)

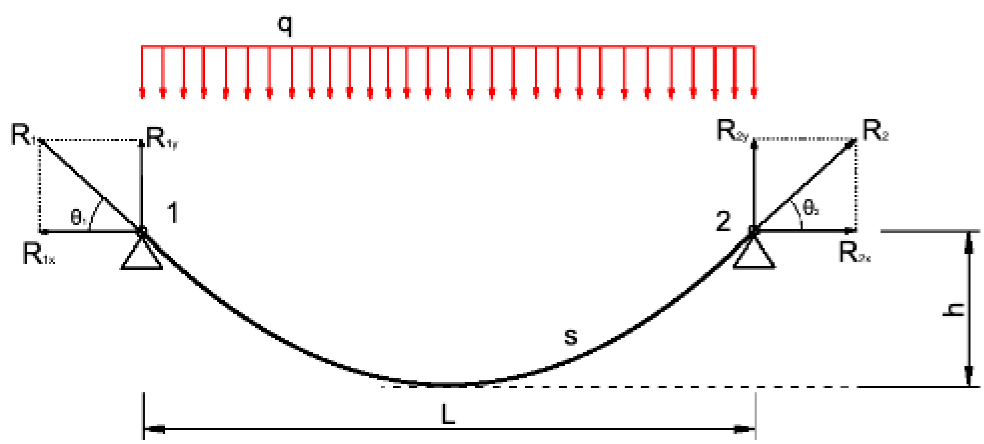

Figure A-3-9. Uniformly Loaded Cable

Horizontal support reaction $\mathbf{R_{1x}}$: .

$$R_{1x} = R_{2x} = \frac{qL^2}{(8h)} = \frac{136(22.5)^2}{(8(1.75))} = 4918 \text{ lbs} \qquad \text{(A-3-1)}$$

Vertical support reaction $\mathbf{R_{1y}}$:

$$R_{1y} = R_{2y} = \frac{qL^2}{2} = \frac{136(22.5)}{2} = 1530 \text{ lbs} \qquad \text{(A-3-2)}$$

Resultant (cable tension):

$$\sqrt{R_{1x}^{2} + R_{1y}^{2}} = \sqrt{R_{2x}^{2} + R_{2y}^{2}} = \sqrt{4918^2 + 1530^2} = 5150 \text{ lbs} \qquad \text{(A-3-3)}$$

For 5/8" 6x19 IWRC cable:

Minimum breaking strength (**MBF**) = 20.60 tons = 41,200 lbs

Min required factor of safety (**FS**) = 6 (applied to the live load per Cal/OSHA and applies to rope/wire rope only per CSO § 1658)

Factor of safety (**FS**) = 41,200/5150 = 8 > 6 ⟸ OK

Connection: 3 wire rope clips (Crosby):

Connection efficiency (**CE**) = 80% (*Falsework Manual*, Table 5-2)

Allowable cable load:

$$\frac{\text{MBF(CE)}}{\text{FS}} = \frac{41{,}200(0.80)}{6} = 5493 \text{ lbs} > 5150 \text{ lbs} \Longleftarrow \text{OK} \qquad \text{(A-3-4)}$$

Cable angle to horizontal (**θ**):

$$\theta = \tan^{-1}\left(\frac{R_{1y}}{R_{1x}}\right) = \tan^{-1}\left(\frac{R_{2y}}{R_{2x}}\right) = \tan^{-1}\left(\frac{1530}{4918}\right) = 17.28° \qquad \text{(A-3-5)}$$

3/4" Shackle working load (per provided data sheet) assumed = 9500 lbs

9500 lbs > 5150 lbs ⟸ OK

<u>Vertical Support Cables</u>

Middle cable reactions = 5'(22.5')(2.06 psf+25 psf) + (22.5')(0.72 plf) = 3060 lbs

For 1/2" 6x19 IWRC cable:

Minimum breaking strength (**MBF**) = 13.30 tons = 26,600 lbs

Min required Factor of safety (**FS**) = 6 (applied to the live load per Cal/OSHA and applies to rope/wire rope only per CSO § 1658)

Factor of safety (**FS**) = 26,600/3060 = 8.7 > 6 ⟸ OK

Connection: 3 wire rope clips (Crosby):

Connection efficiency (**CE**) = 80% (*Falsework Manual*, Table 5-2)

Allowable cable load:

$$\frac{\mathrm{MBF(CE)}}{\mathrm{FS}} = \frac{26{,}600(0.80)}{6} = 3547\ \mathrm{lbs} > 3060\ \mathrm{lbs} \quad \Longleftarrow \quad \mathrm{OK} \qquad \text{(A-3-6)}$$

SafeSpan beam flange clamp (verify capacity with manufacturer's instructions)

3/4" Shackle Working Load (per provided data sheet) assume = 4.75 ton
4.75 ton x 2000 lbs/ton = 9500 lbs > 3060 lbs $\Longleftarrow$ OK

Cable Attachment at Abutment

Cable tension = 5150 lbs (previously calculated)
Angle to horizontal = 17.28º (previously calculated)

SafeSpan end anchor plate (verify capacity with manufacturer's instructions)

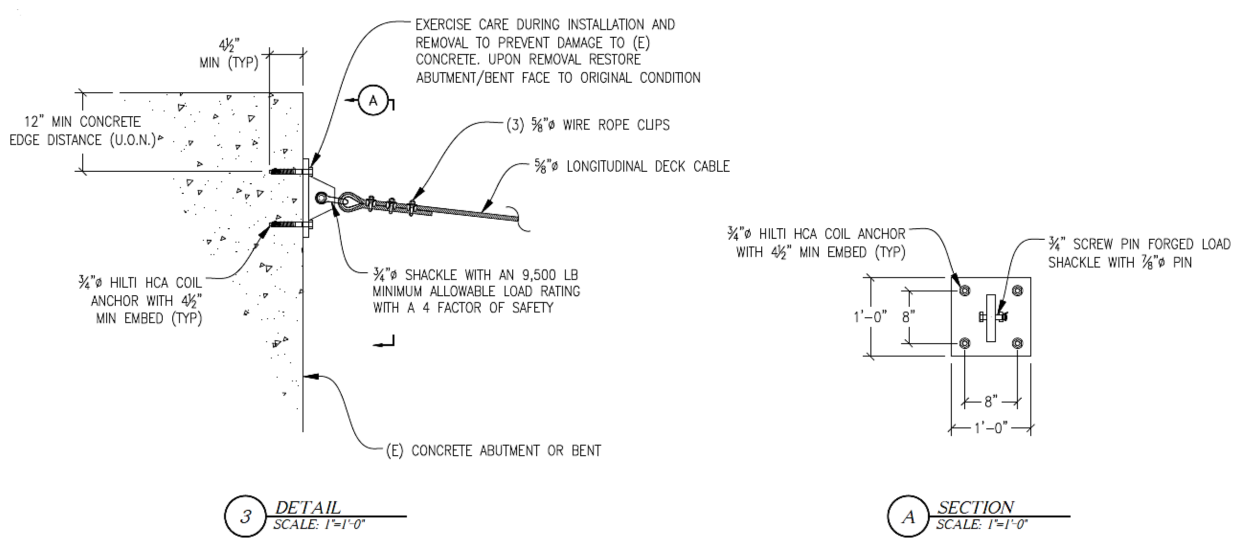

Figure A-3-10. Cable Attachment at Abutment

Anchor Analysis

For four ¾" Hilti HCA coil anchors:

Spacing	=	8" o.c.
Nominal embedment	=	4.5" min.
Edge distance	=	12" min.
Assume **f'c**	=	3000 psi
Vertical load	=	1530 lb (previously calculated)
Horizontal load	=	4918 lb (previously calculated)

Anchor Manufacturer's Instructions

MATERIAL SPECIFICATIONS

1/4-in. HCA manufactured from case hardened AISI 1038 carbon steel with a minimum tensile strength of 100 ksi (690 MPa).
3/8-, 1/2-, 5/8- and 3/4-in. HCA meet the chemical requirements of AISI 1035 carbon steel and are heat treated for a minimum tensile strength of 120 ksi (830 MPa).
Coil is manufactured from carbon steel.
Anchor and coil are zinc plated in accordance with ASTM B633, SC 1.

Figure 1 -
HCA specifications

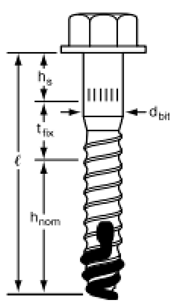

Table 1 - Hilti HCA Coil Anchor specifications

Setting information		Symbol	Units	Nominal anchor diameter				
				1/4	3/8	1/2	5/8	3/4
Nominal bit diameter		d_o	in.	1/4	3/8	1/2	5/8	3/4
Embedment mark[1]		h_s	in.	3/8	5/8	5/8	3/4	1
Anchor Length	min.	ℓ	in.	1-3/4	2-1/4	3	3-1/2	4-1/2
	max.	ℓ	in.	3-1/2	5	7	8	10
Fixture hole diameter		d_h	in.	5/16	7/16	9/16	11/16	13/16
Installation torque		T_{inst}	ft-lb	10	40	80	130	180
Minimum base material thickness		h	in.	the greater of 3 or 1.3 times h_{nom}				

1 Maximum fixture thickness $t = \ell - (h_{nom} + h_s)$

Combined shear and tension loading

$$\left(\frac{N_d}{N_{rec}}\right) + \left(\frac{V_d}{V_{rec}}\right) \leq 1.0$$

Figure A-3-11. Anchor Manufacturer's Instructions

HILTI

Table 2 - Hilti HCA allowable concrete and steel capacity (lb)[1]

Nominal anchor diameter in.	Nominal embedment in.	f'_c = 2,000 psi		f'_c = 4,000 psi		f'_c = 6,000 psi		Allowable steel strength[2]	
		Tension[3]	Shear	Tension[3]	Shear	Tension[3]	Shear	Tension	Shear
1/4	3/4	230	230	325	330	400	400	1,620	835
	1	355	380	500	535	615	655		
3/8	1-1/2	650	850	920	1,205	990	1,475	4,375	2,255
	2	1,005	1,390	1,420	1,965	1,740	2,410		
1/2	2	1,005	1,515	1,420	2,145	1,740	2,625	7,775	4,005
	3	1,845	3,020	2,605	4,270	3,190	5,230		
5/8	2-3/8	1,300	2,175	1,835	3,075	2,250	3,765	12,150	6,260
	3-7/8	2,705	5,000	3,825	7,070	4,685	8,660		
3/4	3-1/4	2,080	3,915	2,940	5,540	3,600	6,780	17,495	9,010
	4-1/2	3,385	6,810	4,790	9,630	5,865	11,705		

1 Allowable concrete capacities based on a safety factor of 4.

2 Steel strength calculated using 0.33 f_{uta} $A_{nominal}$ for tension and 0.17 f_{uta} $A_{nominal}$ for shear.

3 Reduce tension capacity by 20% for HCA Hex Head Bolts that are reused. Coils may not be reused.

Figure A-3-12. Hilti HCA Allowable Concrete and Steel Capacity

Table 4 - Hilti HCA edge distance and anchor spacing guidelines[1,2]

	Load Direction	Critical	Minimum	Influence factor[3]
Spacing	Tension	3.0 h_{nom}	1.0 h_{nom}	f_{AN} = 0.70
	Shear	2.0 h_{nom}	1.0 h_{nom}	f_{AV} = 0.70
Edge distance	Tension	1.5 h_{nom}	0.8 h_{nom}	f_{RN} = 0.75
	Shear ⊥ toward edge[4]	2.5 h_{nom}	1.0 h_{nom}	f_{RV1} = 0.25
	Shear ll or ⊥ away from edge[4]	2.5 h_{nom}	1.0 h_{nom}	f_{RV2} = 0.50

1 For edge and spacing distances between critical and minimum spacing/edge distances, use linear interpolation.

2 Influence factors are cumulative.

3 Influence factor at minimum spacing/edge distance. Influence factor at critical equals 1.0.

4 For shear loads in between perpendicular toward edge and parallel with edge, use the following equation, $f_{RV3} = 0.25 / (\cos \beta + 0.5 \sin \beta)$ for $55° \le \beta < 90°$. For $0° \le \beta < 55°$, use influence factor for shear perpendicular toward edge. See Figure 2.

Figure A-3-13. Hilti HCA Edge Distance and Anchor Spacing Guidelines

Tension and Shear

Maximum anchor plate thickness (**t**), assuming anchor length of 6 inches:

$$t = \boldsymbol{l} - (H_{nom} + h_s) = 6'' - (4.5'' + 1'') = 0.5'' \qquad \text{max (table 1)}$$ A-3-7)

Note:
Verify anchor plate thickness less than or equal to maximum ½" in field

Critical spacing tension	= 3(h_{nom}) = 3(4.5") = 13.5" > 8" (Reduction for spacing required)
Critical edge Dist. tension	= 1.5(h_{nom}) = 1.5(4.5") = 6.75" < 12" (No reduction for edge Dist.)
Critical spacing shear	= 2(h_{nom}) = 2(4.5") = 9" > 8" (Reduction for spacing required)
Critical edge Dist. shear	= 2.5(h_{nom}) = 2.5(4.5") = 11.25" < 12" (No reduction for edge Dist.)

Spacing influence factor for tension:

$$0.7 + 0.3\left(\frac{8 - 4.5}{13.5 - 4.5}\right) = 0.817$$ (A-3-8)

Spacing influence factor for shear:

$$0.7 + 0.3\left(\frac{8 - 4.5}{9 - 4.5}\right) = 0.933$$ (A-3-9)

Allowable loads determined using linear interpolation from manufacture's tables:

Allowable tension load:

$$4\left(\frac{3385 + 4790}{2}\right)(0.817) = 13{,}360 \text{ lbs} > 4918 \text{ lbs} \quad \Longleftarrow \text{OK}$$ (A-3-10)

Allowable shear load:

$$4\left(\frac{6810 + 9630}{2}\right)(0.933) = 30{,}676 \text{ lbs} > 1530 \text{ lbs} \quad \Longleftarrow \text{OK}$$ (A-3-11)

Combined loading:

$$\left(\frac{4918}{13360}\right) + \left(\frac{1530}{30676}\right) = 0.42 < 1 \quad \Longleftarrow \text{OK}$$ (A-3-12)

Existing Bridge Members

Analysis of all individual bridge members is beyond the scope of this example and has not been included.

In accordance with Standard Special Provisions Section 7-1.02K(6)(e), scaffold loads must not exceed the load-carrying capacity of existing bridge members. Member moment and shear demand comparison showing that the scaffolding load demand does not exceed the member moment and shear demands of the available uniform load must be submitted. Members that the available uniform load cannot be applied to, must be analyzed under *AASHTO LRFD Bridge Design Specifications with California Amendments*, latest addition.

The analysis of the existing bridge members using LRFD and LFD may require the assistance of the Bridge Design PE for Bridge Design projects. If the project is a Structure Maintenance and Investigation (SM&I) project, then SM&I would provide support.

Wind Loading Procedure

Wind loads calculated using the provisions of the GSBTW. This structure is a conventional bridge, so the provisions of section 2.3.5.2.1 are used.

Design wind speed when full enclosure is in place = 30 mph (designer service load per submittal).

Design wind speed when scaffold has been vacated = 92 mph (AASHTO LRFD BDS figure 3.8.1.1.2-1).

Wind exposure category C used for this structure.

Wind Pressure

$$P_z = 2.56 \times 10^{-3} V^2 K_Z G C_D K_d \qquad \text{(A-3-13)}$$

V = 30 mph (enclosed structure)

Structure height (**Z**) = 25'

$$K_z(C) = \frac{\left[2.5 \ln\left(\frac{Z}{0.0984}\right) + 7.35\right]^2}{478.4} = \frac{\left[2.5 \ln\left(\frac{25}{0.0984}\right) + 7.35\right]^2}{478.4} = 0.9389 \qquad \text{(A-3-14)}$$

(AASHTO LRFD-BDS 3.8.1.2.1-3)

Gust factor (**G**) = 0.85 (LRFD-BDS 2.3.5.2.3b)

Drag coefficient (**C_D**) = 2.0 (LRFD-BDS table 2.3.5.2.3b-2)

Wind directionality factor (**Kd**) = 0.95 (LRFD-BDS 2.3.5.2.3b)

$P_{Z30} = 2.56\text{x}10^{-3}(30)^2(0.9389)(0.85)(2.0)(0.95) = \underline{3.49 \text{ psf}}$

Area =10.25 sq ft/ft

Force = (3.49 psf)(10.25sq ft/ft)/2 = 17.91 plf (half containment attached to structure)

V = 92 mph (vacated structure)

$P_{Z92} = 2.56\text{x}10^{-3}(92)^2(0.9389)(0.85)(2.0)(0.95) = \underline{32.82 \text{ psf}}$

Area =1 sq ft/ft

Force = 0.6(32.82 psf)(1sq ft/ft) = 19.69 plf ⟸ Governs (Factored load)

Load to ½"Ø lateral stability cable = 25'(19.69 plf) = 492 lbs

For 1/2" 6x19 IWRC cable:

Minimum breaking strength (**MBF**)	=	13.30 tons = 26,600 lbs
Min Factor of safety (**FS**)	=	6 (applied to the live load per Cal/OSHA and applies to rope/wire rope only per CSO 1658)
Factor of safety (**FS**)	=	26,600/492 = > 54 ⟸ OK

Connection: 3 wire rope clips (Crosby)

Connection efficiency (**CE**)	=	80% (*Falsework Manual,* Table 5-2, Wire Rope Connections)

Allowable cable load:

$$\frac{\text{MBF(CE)}}{\text{FS}} = \frac{26{,}600(0.80)}{6} = 3457 \text{ lbs} > 492 \text{ lbs} \Longleftarrow \text{OK} \qquad \text{(A-3-15)}$$