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Division of Engineering Services

Falsework Manual

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The *Falsework Manual* has a long history with Structure Construction starting in the early 1970s with a training program developed by dedicated Bridge Engineers to get Structure Construction staff familiar with the design, review,

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and construction of falsework in order to provide a consistent design and review framework statewide with the ultimate goal of falsework safety. This has been a successful effort through the years and all the engineers, managers, superintendents, foreman, industry partners, editors, and whoever else influenced this manual deserve the highest praise for their tireless efforts and experience in making this manual what it is today – a model and standard for California and the entire United States.

Signed,

RICHARD FOLEY
Deputy Division Chief
Structure Construction
Division of Engineering Services

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Definitions

AISC – American Institute of Steel Construction

AISC Manual – AISC Steel Construction Manual

All other falsework – Represents all falsework except those components that can be classified as heavy-duty steel shoring or steel pipe column falsework with a vertical load capacity greater than 30 kips per leg or column. (*Standard Specifications*, Section 48-2.02B(2), *Falsework – Materials – Design Criteria – Loads*)

Assumed horizontal load – The sum of the actual horizontal loads due to equipment, construction sequence or other causes, and a wind loading. Not to be less than 2% of the total dead load. (*Standard Specifications*, Section 48-2.02B(2), *Falsework – Materials – Design Criteria – Loads*)

Bottom cap – Horizontal member in falsework bent distributing post loads to corbels or pads, typically through wedges and sand jacks.

Bridge camber - Ultimate superstructure deflection curve. (Section 4-2.04, *Camber*)

Cal-OSHA – California Department of Industrial Relations, Division of Occupational Safety and Health (Cal-OSHA)

Camber – An adjustment to the profile of a load-supporting beam or stringer so the completed structure will have the lines and grades shown on the plans. (Section 4-2.04, *Camber*)

Cap – See "Bottom cap" and "Top cap".

Collapse – Term used when the bracing cannot provide sufficient rigidity to prevent unacceptable distortion of the system when subjected to horizontal forces. (Section 6-1, *Introduction*)

Collapsing force – A horizontal force that works to introduce unacceptable distortion to the system. (Section 6-1, *Introduction*)

Come-along – A commonly used term for a lug-all lever or ratchet hoist. (Section 9-3.12D, *Cable Bracing*)

Corbel – Short beam used to distribute the post load across the top of the pad(s). (Section 8-2.01, *Introduction*)

CT – California Department of Transportation (Caltrans)

DES – Caltrans Division of Engineering Services

Design load – Vertical dead and live loads and an assumed horizontal load. (*Standard Specifications*, Section 48-2.02B(2), *Falsework – Materials – Design Criteria – Loads*)

Drag Coefficient (Q) – Used for calculating wind pressure values for "all other falsework". (*Standard Specifications*, Section 48-2.02B(2), *Falsework – Materials – Design Criteria – Loads* and Section 3-3.03C, *Wind Loads on All Other Falsework*)

Falsework – Temporary construction used to support the permanent structure until it becomes self-supporting. (Section 1-1, *Definition of Falsework*)

Falsework bracing system – Those elements designed to resist the "assumed horizontal load" and prevent overturning or collapse. (Section 3-3, *Horizontal Load,* and 6-1, *Introduction*)

Finishing machine – Machine used to finish the concrete during a deck pour. Typically Bidwell. (Section 4-2.02, *Actual Deflection*)

Formwork – (Forms) Used to retain plastic concrete in its desired shape until it has hardened. Designed to resist the fluid pressure of plastic concrete, plus the additional equivalent fluid pressure generated by vibration. May be removed as soon as the concrete hardens, because they do not carry the dead load of the concrete.

FS – Factor of safety

Heavy duty steel shoring – System capable of carrying up to 100 kips per tower leg. Typically, WACO or PAFCO (Section 7-4, *Metal Shoring Systems*)

HQ – Headquarters in Sacramento, California

ICC – International Code Council

ICC-ES – International Code Council Evaluation Service

Joist – Horizontal members that distributes loads from plywood forms on to the stringers.

Limiting length - The length over which a specific falsework pad will actually distribute the post load uniformly at the post location under consideration. (Section 8-2.04D, *Pad Analysis at Exterior Post*)

Load carrying member – Members that control the structural design of falsework.

METS – Materials Engineering and Testing Services within DES

Minimum total design load – 100 psf (*Standard Specifications*, Section 48-2.02B(2), *Falsework – Materials – Design Criteria – Loads*, and Section 3-2.03, *Minimum Total Design Load*)

NDS – 2018 National Design Specification for Wood Construction by the American Wood Council.

Other falsework – See "All other falsework"

Overturning – Term used when the system or element fails by overturning, because the bracing provides sufficient rigidity to the system or element to act as a single rigid unit. (Section 6-1, *Introduction*)

Pad – Timber or concrete members used to distribute the corbel loads or post loads to the soil. (Sections 8-2, *Timber Pads,* and 8-3, *Concrete Pads*)

Pile Bent – Falsework bent where the piles in the foundation extend above the ground and take on the function of the posts. (Section 8-6.01, *Introduction*)

Pony bent – Falsework bent usually erected on, and supported by, a platform constructed on top of primary load carrying members (Section 6-7, *Pony Bent Systems*)

Pork-chop – A common term for wire rope grips due to their shape (Section 9-3.12D, *Cable Bracing*)

Post – Timber or steel member whose primary purpose is to carry axial load from the top cap to the bottom cap, corbel, or pad. In some cases, it may carry combined axial and flexure stress.

Proprietary shoring systems – Systems in which metal components are assembled into modular units that may be stacked to form a series of towers which comprise the vertical load carrying members of the system. (Section 1-7, *System Types*)

Removal – Falsework removal includes lowering the falsework, blowing sand from sand jacks, turning screws on screw jacks, and removing wedges. (*Standard Specification*s, Section 48-2.03D, *Falsework – Construction – Removal*)

Residual camber - Camber remaining in the bridge after it is completed. (Section 4-2.04, *Camber*)

Reviewer – The structure representative or civil engineer registered in the State of California reviewing the falsework shop drawing submittal.

S4S – Surfaced on four sides.

SC – Structure Construction is a subdivision of DES.

Sleeper – Filler strip placed on stringers to prevent joists from bearing on the edge of the stringer flange during deflection and to prevent stringer cantilevered tail upward deflection to hit soffit forms (Sections 4-2.03, *Negative Deflection,* and 4-2.04, *Camber Strips*)

Soffit – The underside or bottom slab of concrete box girder.

Special Locations – Falsework constructed over or adjacent to roadways or railroads open to traffic. (*Standard Specifications*, Section 48-2.02B(4), *Falsework – Design Considerations – Special Locations*)

Stability – Resistance to overturning or collapse of the falsework system as a whole (global) or that portion (local) of the falsework system under consideration. (Section 6-1, *Introduction*)

Stringer - Horizontal member spanning between the falsework bents distributing joist loads to the top caps. Also called beam.

Theoretical effective length - Maximum length over which a falsework pad is capable of distributing the post load uniformly, all other factors being equal. (Section 8-2.02A ,*Effective Bearing Length for Uniform Post Spacing (SYM Formula)*)

Top cap – Horizontal member in falsework bent distributing stringer loads to posts.

Chapter 1: Introduction

1-1 Definition of Falsework

1-1.01 Definition

Falsework: temporary structures used to support the permanent structure until it becomes self-supporting.

1-1.02 Commentary

Falsework includes steel or timber beams, girders, posts, foundations, and any proprietary equipment including modular shoring frames, post shores, and horizontal shoring.

The term “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.

The Standard Specifications 48-2.02B(1), *Temporary Structures – Materials – Design Criteria – General,* provides that the support systems for form panels supporting deck slabs and overhangs on girder bridges are considered to be falsework members and designed as such. On state highway projects all load carrying members, regardless of their location within the supporting system, must meet the design criteria included in the falsework specifications.

1-2 Purpose and Scope

The *Falsework Manual* has been issued by Caltrans, Division of Engineering Services, Structure Construction (SC) to fill the need for a comprehensive design and construction manual devoted exclusively to bridge falsework. Its intended purpose is to provide administrative and technical guidance to the SC field engineers who are in responsible charge of bridge construction on state highway projects. While emphasis is placed on contract administration, it is important to note that materials, design considerations, stress analysis, review criteria, construction, and inspection are covered as well.

This manual includes guidelines, instructions, and procedures which are to be followed on all projects to ensure uniform and impartial contract administration. Project personnel who are responsible for review and authorization of shop drawings and/or inspection of falsework construction are expected to become thoroughly familiar with the contents of this manual.

Proper use of the *Falsework Manual* requires familiarity with the falsework specifications and an understanding of the principles of civil engineering design as applied to bridges and related structures.

1-3 Statement of Department Practice

The *Falsework Manual* sets forth Caltrans' practice for administration of the specifications governing the design and construction of falsework for bridges and related structures on state highway projects.

This manual is not intended to be a contract document. Should there be any conflict between this manual and any contract provisions, the contract provisions must be followed. This is not to say that this manual has no contractual significance. The Standard Specifications 4-1.02, *Scope of Work – Intent,* provides ”the Contract intent is to provide for work completion using the best general practices.” Contractually, the *Falsework Manual* represents Caltrans' opinion on what constitutes "best general practices" within the meaning of this term as it is used in the specifications.

Analytical procedure and review criteria used by SC to evaluate the adequacy of falsework designs, in this manual, are based on more than five decades of continuing study by SC engineering staff of the behavior of individual components of the falsework system, and of the behavior of the system as a whole, as the design loads are applied. These studies, which cover a wide range of typical load combinations, led to the development of simplified methods, which may be used to evaluate the adequacy of complex configurations. Where appropriate, SC has adopted a simplified approach to standardize and facilitate the review process.

For elements of the falsework system that are mathematically indeterminate, the simplified methods and procedures provide reasonably close correlation when compared to results obtained by conventional, rigorous analysis; therefore, they are applicable to the type of falsework encountered on typical bridge projects in California. Occasionally, a situation will arise where analysis using a simplified approach may be inappropriate. In these situations, the design review should include a rigorous analysis to verify stability of the falsework system. The reviewer of the falsework design will be expected to recognize these situations and consult with the SC HQ Falsework Engineer for the procedure to be followed. A falsework system composed of custom-built, multi-tiered structural steel frames or towers is a typical example where a rigorous design analysis would be warranted.

If the contractor's design of an indeterminate element of the falsework system is based on a rigorous analysis as shown by design calculations, and requested in writing by the contractor, the system adequacy will be evaluated by a similar rigorous method of frame analysis. The reviewer should contact the Falsework Engineer at the SC HQ for assistance.

1-4 Practice and Procedural Changes

Information and instructions in the *Falsework Manual* are as current as the publication date. It is expected that changes in practice guidelines and/or procedural direction will occur. Changes will be implemented by issuing dated revisions to the manual. Revisions will be accompanied by instruction or explanation.

To expedite implementation, changes may be applied on an interim basis by issuing falsework memos, which will supersede guidelines in the manual. Falsework memos are to be filed in Appendix C *Falsework Memos* until manual revisions are issued. To ensure that current practice is apparent, interim changes will be noted in the text by a line in the margin.

1-5 Specification Reference

Whenever the term "Standard Specifications", "specifications" or "falsework specifications" appear in this manual, the term is referring to the 2018 edition of the Standard Specifications including the Revised Standard Specifications issued by the California Department of Transportation.

1-6 Design Methodology

Falsework design is based upon Allowable Stress Design (ASD) with members remaining within the elastic range of the material.

The Standard Specifications 48-2.02B(3), *Temporary Structures – Design Criteria – Stresses, Loadings, and Deflection*s, allows the use of design values from the current National Design Specification (NDS) for Wood Construction for identified grades of wood, and from the current *American Institute of Steel Construction (AISC) Steel Construction Manual* for identified grades of steel except for flexural compressive stresses, deflections, and modulus of elasticity.

Falsework over or adjacent to railroads must also comply with the current railroad guidelines.

1-7 System Types

A common falsework system will consist of timber posts, steel caps, timber diagonal bracing, timber or steel stringers, and timber joists. Foundation support is usually provided by timber pads set on the surface of the ground, although poor soil conditions may dictate the use of concrete footings or driven piles to ensure an adequate foundation.

Figure 1-1. Common Falsework System

Pipe posts are commonly used for greater loads and taller falsework. Typical pipe diameters range from 12 to 18 inches. The pipe posts are framed with steel caps, and diagonally braced with small diameter steel rods, reinforcing steel bars, or steel cables.

Figure 1-2. Pipe Post Falsework System

Proprietary shoring systems consist of metal components that may be assembled into modular units and erected in place. When erected, the shoring consists of a series of internally braced towers which support the stringers. The higher capacity of these heavy duty systems will permit longer falsework spans. In most cases, the larger loads associated with heavy duty shoring will require reinforced concrete footings or pile foundations to ensure adequate support.

Figure 1-3. Proprietary Shoring System

Other systems will sometimes be used for complex falsework. These systems can consist of moment resisting frames, steel I-beam posts, steel trusses, and others.

The systems mentioned above can be used as building components for larger and more complex falsework constructions.

1-8 Contractual Relationships

In accordance with contract requirements, the contractor is responsible for the design, construction, and maintenance of falsework. See Standard Specifications 5-1.23B, *Control of Work – Submittals – Action Submittals,* 48-2.01A, *Temporary Structure – Falsework – Summary*, and 48-2.01C(2), *Temporary Structure – Falsework – Shop Drawings*.

The contractor determines the type of falsework to be used and the erection and removal methods to be employed, subject to compliance with the design criteria, safety, and the conditions of use found in the specifications.

Under Department of Transportation policy, review and authorization of the contractor's shop drawings is the responsibility of SC. This responsibility is delegated to the structure representative at the project site. The structure representative or a civil engineer registered in the State of California performs the review. The reviewer is responsible for performing an independent engineering analysis and verifying that the design meets all contract requirements before authorizing the shop drawings.

Authorization of the shop drawings constitutes acceptance of the falsework design by the State and construction details shown on the drawings, and an acknowledgment that the design meets contract requirements.

1-9 Cal-OSHA Requirements

The *Construction Safety Orders* issued by Cal-OSHA include various provisions which apply to the design and construction of falsework or vertical shoring, including falsework or shoring for structures being constructed on state highway projects.

Applicable *Construction Safety Orders,* and the engineer's responsibility with respect to authorization of shop drawings and falsework inspection, is discussed in Chapter 2 *Review of Shop Drawings* and Chapter 9 *Inspection*, respectively.

The term falsework is used in conjunction with both bridge and building construction. However, the temporary supports used in building work are commonly referred to as shores and the support system as shoring.

1-10 State Statutes

The *Streets and Highways Code*, Division 1, Chapter 1, Article 3, Section 137.6 requires that the review and approval of contractor's shop plans for temporary structures in connection with the construction of state highways shall be performed by a civil engineer registered in the State of California. Excerpt from the code are as follows:

- "The design of, the drafting of specifications for, and the inspection and approval of state highway structures shall be by civil engineers licensed pursuant to the Professional Engineers Act (Chapter 7 (commencing with Section 6700), Division 3, Business and Professions Code)."
- "The approval of plans for, and the inspection and approval of, temporary structures erected by contractors in connection with the construction of state highway structures shall also be by such licensed civil engineers."

The Professional Engineers Act (Business and Professions Code), Section 6735, requires all engineering documents be prepared by, or under the responsible charge of, a civil engineer registered in the State of California and be signed and sealed by the engineer. Excerpt from the code are as follows:

- "All civil (including structural and geotechnical) engineering plans, calculations, specifications, and reports (hereinafter referred to as "*documents*") shall be prepared by, or under the responsible charge of, a licensed civil engineer and shall include his or her name and license number."

Chapter 2: Review of Shop Drawings

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2-1 Introduction

This chapter covers Structure Construction (SC) practice with respect to the shop drawing review process. Subsequent chapters cover specific review guidelines, procedures, and explanations to ensure uniform and impartial contract administration.

Review and authorization of the contractor's shop drawings are delegated to the structure representative in responsible charge of structure work at the project site. While any qualified member of the project staff may perform the actual engineering analysis, the structure representative is expected to give personal attention to the review while it is in progress and to give concurrence before the shop drawings are authorized.

The findings from the engineering analysis will be documented in a temporary structure analysis report.

2-2 General Information

The contract requirement for submission of shop drawings should be discussed at the preconstruction meeting, with emphasis on the need for a complete submittal before the review period begins. (See Section 2-4, *Shop Drawing Review*, for information that must be shown in the submittal). With the exception of foundation pads and piles, falsework construction may not begin until the shop drawings are authorized.

When a manufactured product or assembly will be used, the *Standard Specifications*, Section 48-2.01C(1), *Temporary Structures – Falsework – Submittals – General,* require the contractor to furnish catalog data or other descriptive literature showing the manufacturer's recommended safe load carrying capacity, conditions of use, and other information affecting the ability of the product or device to carry the design load. However, such supplemental design information needs to be furnished only if it is requested by the engineer. The contractor should be informed promptly in any case where required technical data is not furnished when the drawings are submitted for review.

On most contracts the engineer will be allowed 20 calendar days to review the shop drawings. For complicated structures, the contract special provisions may establish a longer review period. For falsework near or in railroad right-of-way, the review time is established by the requirements of the railroad company involved. These requirements can usually be found in the *Information Handout* in the bid package.

The drawings may be submitted in increments, and the increments will be reviewed and authorized, provided they are well-defined units of the work, such as individual bridges or portions of bridges that are independent of other portions.

If shop drawings for different units of the work (two or more individual bridges, for example) are submitted at the same time, or if an additional set is submitted for review before review of a previously submitted set has been completed, the contractor must designate the order or sequence in which the sets are to be reviewed. The time allowed for the review of any set in the sequence is not less than the contract time allowed for review of that set, plus 15 calendar days for each additional set. A set consists of 40 or fewer sheets. See also *Standard Specifications*, Section 5-1.23B(2), *Control of Work – Submittals – Shop Drawings*.

When shop drawings are returned for correction, they are to be accompanied by a temporary structure analysis report giving the reason the shop drawings are not acceptable. The report should list the specific deficiencies found, but elaboration is unnecessary. Do not suggest any corrective measures.

2-3 Design Calculations

The *Standard Specifications,* Section 48-2.01C(2), *Shop Drawings,* require the contractor to furnish design calculations supporting the design on the shop drawings. The design calculations must demonstrate the adequacy of the falsework system and show the stresses and deflections in load carrying members. It is not the intent of the specifications to require the contractor to calculate the stress in, and the deflection of, each and every member in the system.

The design calculations furnished by the contractor should not be used during the independent review of the falsework submittal. Any design or construction details which may be shown in the form of sketches on calculation sheets must be shown on the shop drawings as well; otherwise the drawings are not complete. Shop drawings are not to be authorized in any case where it is necessary to refer to calculation sheets for information needed to complete the review and engineering analysis, 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 engineering analysis. However, in the event a member is overstressed or is otherwise inadequate in some respect, reference to the calculations may reveal the reason for the design deficiency.

2-4 Shop Drawing Review

2-4.01 Initial Review

Immediately upon receipt of the first submittal of any set of shop drawings, the structure representative or a civil engineer registered in the State of California will perform an initial review of the documents received. The purpose of the initial review is to ascertain whether the shop drawings and any required supporting data have been submitted.

The goal is to complete the initial review within 2 working days following receipt of a given set of shop drawings. The purpose is to assure a timely notice to the contractor in the event the shop drawings are not complete or required supporting data is missing. Since the only purpose of the initial review is to discover omissions that would prevent completion of a subsequent engineering analysis, neither calculations nor an evaluation of design details is required; thus, completion within 2 working days is a realistic time frame.

Determining whether a particular submittal has sufficient information to perform an independent engineering analysis involves a certain degree of subjectivity, and the engineer will be expected to exercise judgment and common sense when making this determination. The basic requisite is that the shop drawings contain enough information to enable the engineer to verify that the design meets contract requirements. This is accomplished by making an independent engineering analysis. Therefore, if there is not enough information or detail to make an engineering analysis, the shop drawing submittal must be rejected.

Regardless of other considerations, for administrative purposes, the shop drawing submittal must include all the information listed below. If any of the information is omitted, the submittal must be rejected and returned to the contractor for correction and resubmission:

- Size of all load carrying members, including soffit joists, and all transverse and longitudinal bracing, including connections
- Members supporting sloping exterior girders, deck overhangs, and any attached construction walkways
- Design controlling dimensions, including beam length and spacing; post location and spacing; overall height of bents; vertical distance between connectors in diagonal bracing; and similar dimensions that are critical to the design.
- Location and method by which the falsework will be adjusted to final grade
- Unless a concrete placing schedule is shown on the contract plans, the shop drawings must include a superstructure placing diagram showing the proposed concrete placing sequence and/or the direction of pour, whichever one is applicable, and the location of all construction joints. (For relatively simple structures, this requirement may be satisfied by a note on the shop drawings.)
- All openings through the falsework (e.g. traffic, railroad, and pedestrian).Horizontal and vertical clearances must be clearly shown.
- Location of temporary railing

- If the falsework will incorporate a proprietary shoring system, the trade name and rating
- Maximum horizontal distance the piles may be pulled into place under the caps
- Maximum deviation of piles from vertical
- Assumed soil bearing values for pads or footings
- Grade (E-value), species, and type of any structural composite lumber, including manufacturer's tabulated working stress values for the lumber
- Welding standard for any welded members
- If the height of the falsework at any location, measured from the ground line to the bridge soffit, exceeds 14 feet, or if any falsework span exceeds 16 feet, or if openings are provided for vehicular, pedestrian, or railroad traffic; each sheet of the shop drawings must be sealed and signed by a civil engineer registered in the State of California. This includes contractor's standard plans and standard details.
- The shop drawings must be accompanied by the contractor's design calculations, and any other supplemental data required by the falsework design that is needed for an engineering analysis.

The specifications require the contractor to submit design data for any manufactured assembly to be used in the falsework, but only if requested by the engineer. To assure a complete design submittal and thus avoid any unnecessary delay in the review process when a manufactured product or device will be used, the specification requirement should be discussed with the contractor at the preconstruction meeting. The contractor should be informed that if proprietary products of any kind are to be used, the required technical data must accompany the shop drawings when they are first submitted for review.

When reviewing shop drawings pursuant to instructions in this section, submission of complete shop drawings along with all required supporting data is a specific contract requirement that controls the start of the review period. However, while the time period for review of shop drawings does not begin until a complete submittal is received, it is often possible to review portions of the design, which do not depend on the missing information. It is SC practice to expedite the authorization process by reviewing as much of the design as is possible while waiting for the resubmission of shop drawings that have been returned for completion following the initial review.

The initial review may reveal omissions, which are not of such serious consequence as to delay the engineering analysis, but if not corrected will delay authorization. For example, the omission of items such as the erection and removal plan; pad and/or pile design information; the lighting plan, if one is required; and similar information that will eventually be required prior to authorization should be brought to the contractor's attention at the earliest convenient time.

2-4.02 Review

Appendix B, *Falsework Reminder List,* includes a comprehensive listing of items that should be considered and/or investigated during the review of the shop drawings. Prior to the authorization of any shop drawings, the reminder list should be reviewed to verify that no requirement has been overlooked. Section 2-14, *Review of Lighting Plan,* provides information on the review of lighting plans at falsework openings.

2-4.02A Review Procedure when Railroad Company is not Involved

The structure representative or a civil engineer registered in the State of California must review the shop drawings for adequacy and compliance with all contract requirements and all requirements in this manual. An engineering analysis must be performed, see Section 2-4.03, *Engineering Analysis*.

2-4.02B Review Procedure when Railroad Company is Involved

This procedure includes structures adjacent to or over railroad facilities and structures that in any way may impact the railroad or its property.

To ensure that SC is fully informed of all matters relating to falsework over a railroad facility, correspondence to and discussions with the railroad company must be handled by the SC HQ Falsework Engineer. Neither SC field personnel nor contractor personnel are authorized to communicate directly with the railroad. An exception must be approved by the SC Deputy Division Chief.

In most cases, only the shop drawings for the structure span over the railroad tracks will require review and approval by the railroad company. However, when the structure is high enough for adjacent falsework to fall on railroad property or equipment will be used in erecting the falsework, e.g. cranes, which can fall on railroad property, the shop drawings for those adjacent spans must be submitted as well.

Authorization of the shop drawings is contingent upon the shop drawings being satisfactory to the railroad company involved and requires their approval.

The shop drawings must be reviewed for adequacy and compliance with contract requirements in the same manner as all other shop drawings including the engineering analysis (see Section 2-4.03, *Engineering Analysis*). The shop drawings must also be reviewed for compliance with the railroad requirements in the *Information Handout.* The *Standard Specifications,* Section 5-1.20C, *Railroad Relations,* refers to the *Information Handout* for railroad requirements. The Railroad Falsework Check list must also be

completed. The check list can be found on the Temporary Structure Technical Team (Team A) website on the SC intranet.
The "Right-Of-Entry" and "Service Contract" must be fully executed between Caltrans and railroad authorities. Verify that they are not expired because normally they expire after three years. This verification must be done before sending the railroad submittal to the SC HQ Falsework Engineer.

Restricted temporary horizontal and vertical clearances at the railroad tracks require Public Utilities Commission (PUC) approval. Caltrans Right-of-Way requests the approval from the PUC during project development. The structure representative should make certain PUC approval has been granted before the shop drawings are authorized.

The following information must be on the first sheet of the shop drawings. This information can usually be found in the *Information Handout*:

- DOT #
- RR Milepost
- Subdivision
- Closest City
- Longitude
- Latitude:

See also *Falsework Manual*, Section 4-12.06F, *Shop Drawings Over or Adjacent to Railroad,* for additional information.

When the engineering analysis has been completed and the structure representative is satisfied that all contract requirements and railroad requirements have been met, the structure representative must submit PDFs of the following by email to the SC HQ Falsework Engineer:

- Shop drawings (reviewed, but not authorized)
- Review calculations
- Contractor's calculations
- Manufacturer's catalog data for manufactured assemblies
- Railroad Falsework Check list
- General plan of the contract plans

The Railroad Relations section in the *Information Handout* must be reviewed before the shop drawings are sent to the SC HQ Falsework Engineer to ensure that all applicable requirements have been met.

The email must state that the shop drawings have been reviewed, an engineering analysis has been completed, and the submittal is satisfactory.

The SC HQ Falsework Engineer will review the submittal and subsequently forward the submittal to the railroad for approval.

When the SC HQ Falsework Engineer is informed by the railroad company that the shop drawings are satisfactory, the structure representative will be notified by email including the railroad approval.

The railroad approval may include conditions. These conditions must be addressed before authorizing the shop drawings.

Upon notification and after any applicable railroad approval conditions have been satisfied, the structure representative may authorize the drawings as described in Section 2-6.01, *Shop Drawing Authorization*.

It is emphasized that shop drawings for structures over railroad facilities are not to be authorized until the SC HQ Falsework Engineer has notified the structure representative that the design is acceptable and has been approved by the railroad company.

2-4.03 Engineering Analysis

When the contractor submits shop drawings, which are required to be stamped and signed by a civil engineer registered in the State of California, it is SC practice to perform an independent engineering analysis.

Upon completion of the engineering analysis of the shop drawings, the structure representative or the civil engineer registered in the State of California performing the engineering analysis will present the findings in a temporary structure analysis report. This includes sealing and signing the report in accordance with the *Streets and Highways Code*, Section 137.6, and the Professional Engineers Act (Business and Professions Code), Section 6735. See Section 1-10, *State Statutes*.

This report is to be completed for rejected and authorized shop drawings. The report should contain a brief chronological record of the pertinent dates relating to the submission, review, rejection, and authorization, including the number of review days. This does not replace the chronological record in Section 2-9, *Chronological Record of Shop Drawing Review*. The structure representative must transmit the report to the contractor, see Section 2-10, *Transmittal to the Contractor*. An example of the report is shown in Section 2-4.04, *Sample Temporary Structure Analysis Report*.

When the shop drawings cannot be authorized, complete the temporary structure analysis report and list the reason(s) the shop drawings are not acceptable. Elaboration is unnecessary. Do not suggest any corrective measures. Prior to sending the report to the contractor, contact the falsework design engineer 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 are authorized, complete the temporary structure analysis report. The report must include the following paragraphs:

- "The *(insert type of review completed, i.e. falsework, trenching and shoring, column guying)* shop drawings for *(identify specific location)* of the (*Bridge name, Br. No.),* are found acceptable based upon an independent engineering analysis and are authorized to the extent provided in the *Standard Specifications* Section 5-1.23, *Submittals*."
- "Your attention is directed to your responsibilities pursuant to *Standard Specifications* Sections 5-1.23, *Submittals*, 7-1.04, *Public Safety*, and (*insert appropriate Standard Specification reference, i.e. Standard Specifications*, Section 48, *Temporary Structures*), and to applicable requirements of the *Construction Safety Orders*."
- "You are reminded that *(insert type of temporary structure, i.e. falsework, shoring, etc.)* construction must conform to the authorized shop drawings, 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 temporary structure, i.e. falsework, shoring, etc.)* will support the loads imposed without excessive settlement or joint take-up beyond that shown on the authorized *(insert type of temporary structure, i.e. falsework, shoring, etc.)* shop drawings."

The sample temporary structure analysis report in Section 2-4.04 *Sample Temporary Structure Analysis Report* can be used as a template.

2-4.04 Sample Temporary Structure Analysis Report

(see next page)

Temporary Structure Analysis Report

Insert Date

Project Information

Contract Number
Dist-Co-Rte-PM
Bridge name
Br. No.

Type of structure reviewed: *(insert falsework, trenching and shoring, column guying)*

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)*

Review completed: *(date)*

Elapsed review time: *(days)*

Introduction:

This report presents the results of an independent engineering analysis for the *(insert type of review completed, i.e. falsework, trenching and shoring, column guying)* at (*identify specific location i.e. Frame 1, stage 1 etc.)*

Discussion:

Rejection – This portion of the report would describe specific deficiencies found with the shop drawings that would be cause for rejection i.e. The following members have been found to be overstressed:

W36x240 stringer in span FW5-6 is overstressed in bending
Post in bent FW5 overstressed in compression

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

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

Conclusion:
<u>*Rejection:*</u>

The *(insert type of review completed, i.e. falsework, trenching and shoring, column guying)* shop drawings for *(identify specific location)* of the (*bridge name, Br. No.),* are rejected based upon an independent engineering analysis. The deficiencies are listed above.

<u>*Authorization (the paragraphs below must be included):*</u>

"The *(insert type of review completed, i.e. falsework, trenching and shoring, column guying)* shop drawings for *(identify specific location)* of the (*bridge name, Br. No.),* are found acceptable based upon an independent engineering analysis and are authorized to the extent provided in the *Standard Specifications* Section 5-1.23, *Submittals*."

"Your attention is directed to your responsibilities pursuant to *Standard Specifications,* Sections 5-1.23, *Submittals*, 7-1.04, *Public Safety*, and (*insert appropriate Standard Specification reference, i.e. Standard Specifications,* Section 48, *Temporary Structures*), and to applicable requirements of the *Construction Safety Orders*."

"You are reminded that *(insert type of temporary structure, i.e. falsework, shoring, etc.)* construction must conform to the authorized shop drawings, 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 temporary structure, i.e. falsework, shoring, etc.)* will support the loads imposed without excessive settlement or joint take-up beyond that shown on the authorized *(insert type of temporary structure, i.e. falsework, shoring, etc.)* shop drawings."
If you have any questions regarding this report, please contact the structure representative at (XXX) XXX-XXXX.

*(Signature)*________________
LOREN N, BRIDGE, P.E.
Structure Representative
Structure Construction

2-5 Review Duration

As previously noted, the time allowed by the contract for the shop drawing review is 20 calendar days, or more for complicated structures or when railroad approval is required (see Section 2-2, *General Information*).

Regardless of the time allowed, the review period begins when the shop drawings and required supporting information have been received from the contractor and ends when the shop drawings are authorized or rejected. The engineer is not responsible for time taken by the contractor to make necessary revisions or corrections to the shop drawings.

Revised submittals receive the same number of review days as the original submittal. Although the review time for a resubmittal is the same as the original, it is SC practice to make an effort to review the resubmitted shop drawings in less than the specified time if the revisions are minor. As a guide, a resubmittal will be viewed as a minor revision when it does not require another engineering analysis to verify system adequacy.

The actual duration of the review can be critical from an administrative standpoint, since the contract provides for additional time and compensation to offset any time lost due to delays attributable to the failure to complete the shop drawing review within the time allowed. In the event the contractor is delayed, the additional time and compensation due are determined in accordance with the delay provisions of the contract.

2-6 Shop Drawing Authorization and Rejection

2-6.01 Shop Drawing Authorization

Prior to the authorization of any shop drawings, the shop drawings must be reviewed and an engineering analysis done according to Section 2-4, *Shop Drawing Review,* to verify that no requirement has been overlooked.

The shop drawings must be authorized by placing the authorization stamp on each sheet, see Figure 2-1, *Authorization Stamp*. The stamp must be signed and dated by the structure representative or the reviewer.

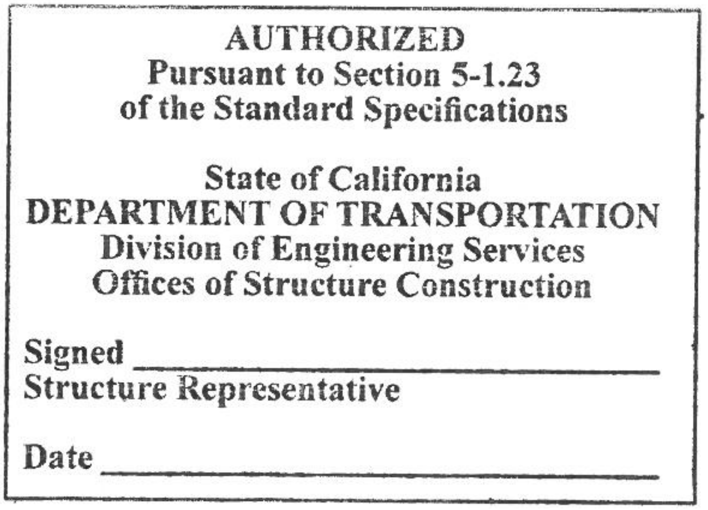


Figure 2-1. Authorization Stamp.

Shop drawings may be authorized using an electronic authorization stamp, see Figure 2-2, *Electronic Authorization Stamp*. The stamp must be signed by the structure representative or the reviewer. The signature may be electronic. The procedure to obtain an electronic signature is available from the SC HQ Falsework Engineer and can be found on the Temporary Structure Technical Team (Team A) intranet website.

Figure 2-2. Electronic Authorization Stamp.

2-6.01A Authorization when Railroad Company is not Involved

The shop drawings may be authorized when:

- The structure representative or a civil engineer registered in the State of California has completed the shop drawing review and engineering analysis.
- The structure representative is satisfied that the design meets all contract requirements.

See Section 2-4, *Shop Drawing Review;* Section 2-4.02A, *Review Procedure when Railroad Company is not Involved*; and Section 2-4.03, *Engineering Analysis.*

The structure representative must stamp each sheet as described in Section 2-6.01, *Shop Drawing Authorization*.

2-6.01B Authorization when Railroad Company is Involved

This authorization includes structures adjacent to or over railroad facilities and structures that in any way may impact the railroad or its property, see Section 2-4.02B, *Review Procedure when Railroad Company is Involved*.

It is emphasized that authorization of shop drawings is contingent upon the shop drawings being satisfactory to the railroad company involved. They are not to be authorized until the SC HQ Falsework Engineer has notified the structure representative that the design is acceptable and has been approved by the railroad company.

The shop drawings may be authorized when:

- The structure representative or a civil engineer registered in the State of California has completed the shop drawing review and engineering analysis.
- The structure representative is satisfied that the design meets all contract requirements.
- The structure representative has been notified of the railroad approval by the Falsework Engineer.
- The structure representative has addressed any conditions in the railroad approval.

See Section 2-4.02B, *Review Procedure when Railroad Company is Involved*, and Section 2-4.03, *Engineering Analysis*.

The structure representative or the reviewer must stamp each sheet as described in Section 2-6.01, *Shop Drawing Authorization*.

2-6.02 Shop Drawing Rejection

When the shop drawings cannot be authorized, stamp the date on each sheet of the shop drawings when the review stopped. See *Standard Specifications,* Section 5-1.23B(2), *Shop Drawings*.

Describe the deficiencies in the temporary structure analysis report, see Section 2-4.03, *Engineering Analysis*. For clarity redline clouds may be used on the shop drawings.

2-7 Cal-OSHA Requirements

Construction Safety Orders, Chapter 3.2, Subchapter 2, Article 2, *Permits--Excavations, Trenches, Construction and Demolition and the Underground Use of Diesel Engines in Work in Mines and Tunnels*, requires the contractor to obtain a permit for the "Erection and placement of scaffolding, vertical shoring, or falsework intended to be more than 36 feet high when completed." This requirement will apply to all falsework, which exceeds 36 feet, measured from the lowest point of surrounding grade or ground level to the bridge soffit.

Obtaining the permit required pursuant to Article 2 is the contractor's responsibility. Upon application, a permit will be issued for a specific project; however, as a business practice many bridge contractors obtain an annual permit to cover all their work. Annual permits are valid for the period from January 1 to December 31.

Although the structure representative has neither the authority nor the duty to enforce the article mentioned above, as a matter of practice, when a permit is required, the fact that the contractor has a valid permit should be verified before the shop drawings are authorized. The date of verification should be noted in the daily report and the chronological record, see Section 2-9, *Chronological Record of Shop Drawing Review*.

2-8 Revisions to Shop Drawings

2-8.01 Revisions to Rejected Shop Drawings

The *Standard Specifications,* Section 5-1.23B(1), *Control of Work – Action Submittals – General*, allow the same number of days for a revised submittal as for the original submittal. See Section 2-5, *Review Duration,* for additional information.

The contractor must show the revision number on the revised shop drawings and uniquely number each revised detail, and describe and date the revisions in a legend, see *Standard Specifications,* Section 5-1.23B(2), *Shop Drawings*.

2-8.02 Revisions to Authorized Shop Drawings

The specifications contemplate the possibility of the contractor submitting a revised design after the original design has been reviewed and authorized. The revised design is to be submitted as a new submittal per *Standard Specifications,* Section 5-1.23B(2), *Shop Drawings,* and will be reviewed pursuant to applicable specification requirements and the review practice and procedures discussed in this chapter.

2-9 Chronological Record of Shop Drawing Review

2-9.01 Introduction

A chronological record, or log, showing all pertinent dates relating to the submission, review, and authorization of shop drawings is required for each structure in the contract.

Normally, the first entry will be the date the shop drawings are first received. If, however, topics having significance with respect to the design are discussed prior to the first submittal, the discussion should be noted, and the log started.

This log will include, but not limited to:

- The date the shop drawings were first received.
- The date(s) the contractor is notified of required revisions, including the reason(s) the review could not continue and/or why the shop drawings had to be returned.
- The date(s) revised shop drawings were received.
- The date(s) and subject matter covered in conversations, letters, and emails, relating to the review.
- The date the shop drawings were sent to the SC HQ Falsework Engineer for railroad approval and when railroad approval was received.
- The date of authorization; the date the authorized shop drawings were forwarded to SC HQ.

When entries are properly made, the time taken for the engineer's review should be readily apparent. Other functional units will use this information to establish review times, especially for complex bridges and those involving other agencies, such as the railroad.

Make a notation in the log of the date that falsework for a given structure becomes the controlling operation on the project, and the date on which it is no longer controlling. Be specific as to the activity that is controlling, such as preparation of drawings by the contractor, review by the State or railroad company, erection, etc. In some situations, particularly where a Critical Path Method (CPM) analysis has not been made, it may be difficult to ascertain whether falsework is, or is not, on the critical path. If the actual controlling operation is not evident but it appears that the falsework review (or construction) may affect other aspects of the project, the structure representative should note this fact in the log.

Entries in a chronological record are not in lieu of similar information shown in construction daily reports. The diary entry should give detailed information, whereas the chronological record should list only the dates, identification of subject, and the people involved. The level of details will depend on the complexity of the project. An example of a chronological record is shown in Section 2-9.02, *Example of Chronological Record of Falsework Review*.

2-9.02 Example of Chronological Record of Falsework Review

Chronological Record of Falsework

Review *Insert Date*

Project Information

Contract Number
Dist-Co-Rte-PM
Bridge name
Br. No.

Type of structure reviewed: *(insert falsework, trenching and shoring, column guying)*

Chronological Record:

Event No.	Date	Comments and/or References
1	4-01-2017	Contractor submitted shop drawings and calculations for frames I & 2. (See attached copy of transmittal letter dated 4-01-2017).
2	4-04-2017	Discussed FW submittal with contractor's engineer. Submittal incomplete, need pedestrian protection details, catalog data, etc. (See SR daily report).
3	4-08-2017	Received additional FW data. (See copy of transmittal letter dated 4-08-2017) **Falsework review period begins for frames 1 & 2.**
4	4-10-2017	Contractor set pads for spans 1 & 2 in frame 1. (See ASR daily report).
5	4-12-2017	Contractor revised beam sizes in FW spans 1-4 & 1-5. (See attached copy of transmittal letter dated 4-12-2017). This is revision 1.
6	4-18-2017	Frame 1 (incl. Rev. I) and frame 2 (except span 7 over RR) approved. (See attached copy of analysis report to contractor).
7	4-18-2017	Frame I (incl. Rev. 1) drawings with contractor’s and SR calculations and analysis report transmitted to SC HQ. Frame 2 drawings with contractor’s and SR calculations and RR check list sent to SC HQ Falsework Engineer for review and transmittal to RR. (See attached email to SC HQ Falsework Engineer).
8	4-22-2017	Contractor revised lateral connections at FW bents 1-6, I-7 & I-8. (See attached copy of transmittal letter). This is Revision 2.
9	4-23-2017	Rev. 2 approved and sent to SC HQ. (See attached copy of analysis report to contractor).
10	5-07-2017	Contractor requested status of RR review. (See SR daily report).
11	5-17-2017	RR approved span 7. Advised contractor. (See SR daily report). Received confirming email from SC HQ Falsework Engineer regarding RR approval. Sent analysis report approving span 7 to contractor. (See attached copy of RR approval, email from SC HQ Falsework Engineer, and analysis report).

2-10 Transmittal to the Contractor

2-10.01 Authorized Submittals

Write and sign a transmittal letter to the contractor per your office or district protocol. The transmittal letter should refer to the temporary structure analysis report for findings and decision. Attach the following to the transmittal letter:

- The authorized shop drawings with the SC authorization stamp and the structure representative's or reviewers' signature and date, see Section 2-6.01, *Shop Drawing Authorization*
- The temporary structure analysis report with the professional engineer's stamp, signature, and date, see Section 2-4.03, *Engineering Analysis*

Send the transmittal letter and the attachments to the contractor.

For falsework submittals where the shop plans are not required to be stamped by a civil engineer registered in the State of California (see *Standard Specifications,* Section 48-2.01C(2), *Shop Drawings*), the temporary structure analysis report is not required (see Section 2-4.03, *Engineering Analysis*), the authorization is stated in the transmittal letter and the letter must include the following paragraphs:

- "The *(insert type of review completed, i.e. falsework, trenching and shoring, column guying)* shop drawings for *(identify specific location)* of the (*Bridge name, Br. No.)* have been reviewed and are authorized to the extent provided in *Standard Specifications,* Section 5-1.23, *Submittals*."
- "Your attention is directed to your responsibilities pursuant to *Standard Specifications,* Sections 5-1.23, *Submittals*, 7-1.04, *Public Safety*, and (*insert appropriate Standard Specification reference, i.e. Standard Specifications,* Section 48, *Temporary Structures*), and to applicable requirements of the *Construction Safety Orders*."
- "You are reminded that *(insert type of temporary structure, i.e. falsework, shoring, etc.)* construction must conform to the authorized shop drawings, 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 temporary structure, i.e. falsework, shoring, etc.)* will support the loads imposed without excessive settlement or joint take-up beyond that shown on the authorized *(insert type of temporary structure, i.e. falsework, shoring, etc.)* shop drawings."

The transmittal letter is not an engineering document and therefore should not be stamped with a professional engineer's stamp.

For sample transmittal letter, See Section 2-10.03, *Sample Transmittal Letters*.

2-10.02 Rejected Submittals

Write and sign a transmittal letter to the contractor per your office or district protocol. The transmittal letter should refer to the temporary structure analysis report for findings and decision. Attach the following to the transmittal letter:

- The rejected shop drawings with the date the review stopped stamped on each sheet, see Section 2-6.02, *Shop Drawing Rejection*
- The temporary structure analysis report with the professional engineer's stamp, signature, and date, see Section 2-4.03, *Engineering Analysis*

Send the transmittal letter and the attachments to the contractor.

For falsework submittals where the shop plans are not required to be stamped by a civil engineer registered in the State of California (see *Standard Specifications,* Section 48-2.01C(2), *Shop Drawings*), the temporary structure analysis report is not required (see Section 2-4.03, *Engineering Analysis*).

In this case, state the rejection in the transmittal letter and list the reason(s) the shop drawings are not acceptable. Elaboration is unnecessary. Do not suggest any corrective measures. Prior to sending the transmittal letter to the contractor, contact the falsework design engineer by phone or in person to discuss the reason(s) for rejecting the submittal. Document this discussion in the chronological record (see Section 2-9, *Chronological Record of Shop Drawing Review*) and the daily report.

The transmittal letter is not an engineering document and therefore should not be stamped with a professional engineer's stamp.

For sample transmittal letter, See Section 2-10.03, *Sample Transmittal Letters*.

2-10.03 Sample Transmittal Letters

(see next page)

Transmittal Letter – With Temporary Structure Analysis Report – Authorized and Rejected

STATE OF CALIFORNIA-------CALIFORNIA STATE TRANSPORTATION AGENCY Gavin Newsome, Governor

DEPARTMENT OF TRANSPORTATION

<Your Office Address>
<Your Office Phone>

Date: <Date>

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

<Contractor Name>
<Contractor Address>

Dear <Responsible Person>,

The falsework shop drawings for Camarillo Overhead and Separation (Widen), Bridge No. 52- 16, as revised December 1, 2017, have been reviewed. Your attention is directed to the attached Temporary Structure Analysis Report.

Sincerely,

Resident Engineer

Attachments:

(Authorized or rejected) falsework shop drawings
Temporary Structure Analysis Report

Transmittal Letter – Without Temporary Structure Analysis Report - Authorized

STATE OF CALIFORNIA-------CALIFORNIA STATE TRANSPORTATION AGENCY Gavin Newsome, Governor

DEPARTMENT OF TRANSPORTATION

<Your Office Address>
<Your Office Phone>

Date: <Date>

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

<Contractor Name>
<Contractor Address>

Dear: <Responsible Person>,

The falsework shop drawings Camarillo Overhead and Separation (Widen), Bridge No. 52-0016, as received on December 1, 2017, have been reviewed and are authorized to the extent provided in *Standard Specifications,* Section 5-1.23, *Submittals*.

Your attention is directed to your responsibilities pursuant to *Standard Specifications,* Sections 5-1.23, *Submittals*, 7-1.04, *Public Safety*, and 48, *Temporary Structures*, and to applicable requirements of the Construction Safety Orders.

You are reminded that falsework construction must conform to the authorized shop drawings, 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 falsework will support the loads imposed without excessive settlement or joint take-up beyond that shown on the authorized falsework shop drawings.

Sincerely,

Resident Engineer

Attachments:
Authorized falsework shop drawings

Transmittal Letter – Without Temporary Structure Analysis Report - Rejected

STATE OF CALIFORNIA-------CALIFORNIA STATE TRANSPORTATION AGENCY Gavin Newsome, Governor

DEPARTMENT OF TRANSPORTATION

<Your Office Address>
<Your Office Phone>

Date: <Date>

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

<Contractor Name>
<Contractor Address>

Dear <Responsible Person>,

The falsework shop drawings Camarillo Overhead and Separation (Widen), Bridge No. 52-0016, as received on December 1, 2017, have been reviewed and are rejected. The deficiencies are listed below:

The following members have been found to be overstressed:

W14x120 stringer in span 1 is overstressed in bending
Post in bent FW2 overstressed in compression

If you have any questions regarding this report, please contact the structure representative at (XXX) XXX-XXXX.

Sincerely,

Resident Engineer

Attachments:

Rejected falsework shop drawings

2-11 Submittal Guidelines

2-11.01 Introduction

This section is intended as a brief guideline showing where to send shop drawing submittals, how to name files, and how to label email subject lines.

2-11.02 Submittals Where Railroad is Not Involved

Of the documents listed below, retain one copy in the job file and send one copy to SC HQ for retention in VISION and emergency response:

- Authorized shop drawings
- Temporary structure analysis report
- Reviewer's engineering analysis calculations
- Contractor's calculations
- Manufacturer's catalog data for manufactured assemblies

Follow these guidelines for files and emails when sending to Sacramento:

- Scan each document to PDF
- Name each file as shown in Section 2-11.04, *File Naming Convention*
- Send by email to: SC Office Associates@dot.ca.gov
- Send copy of email to SC HQ Falsework Engineer
- In the email subject line enter:
    - Contract # - Bridge Name (Bridge No.) - Submittal Type

2-11.03 Submittals When Railroad is Involved

To obtain approval from the railroad send the following documents to the SC HQ Falsework Engineer:

- Shop drawings (reviewed, but not authorized)
- Reviewer's calculations
- Contractor's calculations
- Manufacturer's catalog data for manufactured assemblies
- Railroad falsework check list
- General Plan of the contract plans

Follow these guidelines for files and emails when sending to Sacramento:

- The following information must be on the first sheet of the shop drawings:
  - DOT #
  - RR Milepost
  - Subdivision
  - Closest City
  - Longitude
  - Latitude
- Scan each document to PDF.
- Name each file as shown in Section 2-11.04, *File Naming Convention.*
- Send by email to SC HQ Falsework Engineer the same day authorization is sent to the contractor.
- In the email subject line enter:
  - Contract # - Bridge Name (Bridge No.) - Submittal Type.

After the railroad has approved the submittal and the structure representative has authorized the shop drawings, retain one copy of the following documents in the job file and send one copy to SC HQ for retention in VISION and emergency response:

- Authorized shop drawings
- Temporary structure analysis report
- Engineering analysis calculations
- Contractor's calculations
- Manufacturer's catalog data for manufactured assemblies
- Railroad approval
- Railroad falsework check list

Follow these guidelines for files and emails when sending to Sacramento:

- Scan each document to PDF.
- Name each file as shown in Section 2-11.04, *File Naming Convention.*
- Send by email to: SC Office Associates@dot.ca.gov.
- Send copy of email to: SC HQ Falsework Engineer.
- In the email subject line enter:

- Contract # - Bridge Name (Bridge No.) - Submittal Type.

2-11.04 File Naming Convention

Name each scanned PDF as shown for ease of archiving and retrieval in VISION. The short-abbreviated naming convention is chosen for easy searching in VISION. Due to the large number of files stored in VISION, it is important that all file names are consistent throughout the state.

The naming conventions were developed to also cover other submittal types.

The abbreviations shown below cover most submittals. There may be cases where a proper abbreviation is not listed below. In this case consult the SC HQ Falsework Engineer about the proper abbreviation to use.

Naming convention:

- Contract # / Bridge # / Stage, Frame, etc / Submittal Type / Component
- Note: slash “ / ” indicates space

Abbreviations for **Stage, Frame, Span, or Location**:

Fr = Frame

Sp = Span

St = Stage

A# = Abutment and number

B# = Bent and number

Abbreviations for **Submittal Type**:

FW =	Falsework
Demo =	Bridge Removal
Guy =	Column/wall Rebar Guying Plan
Pile =	Pile Installation (e.g. CIDH piles)
Scaff =	Scaffold
SH =	Shoring and Excavation
Trest =	Trestle
TS =	Temporary Support

Abbreviations for **Component**:

AR =	Temporary Structure Analysis Report *(Includes Transmittal Letter)*
AP =	Adjustment Plan
Cat =	Manufacturer's catalogue, data sheet
CCalc =	Contractor's Calculations
LP =	Lighting Plan
SCalc =	SR Calculations
SD =	Shop Drawings (plans)
PP =	Placement Plan (e.g. pile placement plan written procedure)
PSD =	Permanent steel deck forms
RP =	Removal Plan (written procedure). Sometimes the removal plan is written on the shop drawings. In this case use SD
RR =	Railroad approval
TL =	Transmittal Letter with authorization *(Only needed when AR is not required)*

Example of file names:

04-120004 34-0120L Fr1 FW AR.pdf

04-120004 34-0120L Fr1 FW SD.pdf

04-120004 34-0120L Fr1 FW CCalc.pdf

04-120004 34-0120L Fr1 FW SCalc.pdf

04-120004 34-0120L Fr1 FW Cat.pdf

2-12 Responsibilities

This section is intended as a brief guideline showing the responsibility of the person involved in the shop drawing review.

2-12.01 Structure Representative

Structure representative has the following responsibilities pertaining to falsework submittals:

- Performs or oversees review of the independent engineering analysis.
- Confirms that the submittal complies with contract requirements and railroad requirements where railroad is involved in the project.
- Corresponds with contractor (rejects/authorizes).

- Keeps track of review times.
- Prepares seals and signs temporary structure analysis report.
- Confirms that shop drawings, materials incorporated, and construction methods meet contract requirements and the best general practices represented in the *Falsework Manual.*

2-12.02 Assistant Structure Representative

Assistant structure representative has the following responsibilities pertaining to falsework submittals:

- Performs review and independent engineering analysis.
- Keeps track of review times.
- Prepares seals and signs temporary structure analysis report.

2-12.03 SC HQ Falsework Engineer

SC HQ Falsework Engineer has the following responsibilities pertaining to falsework submittals:

- Performs a cursory review of all shop drawings and calculations for falsework adjacent to or over railroads and forwards the shop drawings and calculations to the railroad for their review and acceptance.
- Spot checks shop drawings and calculations. The shop drawings to be spot checked are selected at random with the objective of ascertaining compliance with current falsework directives and practices.
- Acts as consultant to the structure representative and field engineers and provides guidance with complicated falsework problems and resolves questions involving policy and practice.
- Acts as the temporary structure liaison between the project and the railroad.

2-13 Field Review of Falsework

2-13.01 Introduction

Falsework erected by contractors during construction of state highway bridges and related structures needs to remain safe, stable, and serviceable throughout its' design life. Failure of falsework can be catastrophic. SC practice is to incorporate all contract requirements, experience, and best general practice to prevent falsework failures. SC staff will have the following responsibilities as a minimum to ensure a safely constructed system.

2-13.02 Structure Representative & Assistant Structure Representative

Responsibilities of the structure representative and the assistant structure representative:

- Verify that falsework is constructed as per the authorized shop drawings.
- Verify that all pertinent load tests are performed and properly documented.
- Verify that the falsework construction meets all Cal-OSHA applicable safety orders.
- Verify that the falsework is inspected and certified by the contractor’s falsework designer or their authorized representative pursuant to the requirements in *Construction Safety Orders,* Article 1717, the *Standard Specifications,* Section 48-2.01C(2), *Shop Drawings*, and the contract Special Provisions.
- Verify that jacking and displacement monitoring systems are authorized and in place prior to jacking.
- Work closely with the contractor’s falsework foreman to coordinate all aspects of erecting, grading, and removing the falsework safely.

2-13.03 Bridge Construction Engineer

Responsibility of the Bridge Construction Engineer:

- Perform a field review of falsework installations, together with the structure representative, before concrete is placed. Refer to the authorized shop drawings during this review.

2-13.04 Area Construction Manager

Responsibility of the Area Construction Manager:

- Periodically perform a field review of falsework installations with Bridge Construction Engineers in their areas. Refer to the authorized shop drawings during this review.

2-13.05 SC Oversight Engineer

Responsibility of the SC oversight engineer:

- Perform a field review of falsework installations, together with local agency structure representative, before concrete is placed, see BCM D-1.04, *Administration of Local Agency Projects.* Refer to the authorized shop drawings during this review.

2-14 Review of Lighting Plan

Standard Specifications, Section 48-2.01D(2)(b), *Quality Control – Falsework Lighting*, Section 48-2.02C, *Temporary Structures – Materials – Falsework Lighting*, and Section 48-2.03E, *Temporary Structures – Construction – Falsework Lighting,* state the requirements for pavement and portal lighting at traffic openings. Any project specific requirements will be shown on the contract plans and/or included in the special provisions.

The *Standard Specifications,* Section 48-2.01C(1), *Falsework - Submittals – General,* require the contractor to submit a lighting plan before starting construction of falsework containing openings for vehicular traffic, pedestrians, or railroad. The lighting plan must be authorized by the engineer before falsework construction at the traffic opening commences. The lighting plan is not part of the falsework shop drawing submittal covered by *Standard Specifications,* Section 48-2.01 C(2), *Falsework - Submittals – Shop Drawings.* It is a separate action submittal, which is reviewed and authorized pursuant to *Standard Specifications,* Section 5-1.23B, *Action Submittals*. However, if the lighting plan is shown on the shop drawings, authorization of the shop drawings will constitute authorization of the lighting plan as well.

The lighting plan should be reviewed from the viewpoint of public traffic, and for employee safety during routine maintenance work as well. The *Standard Specifications,* Section 48-2.03E(1), *Temporary Structures – Construction – Falsework Lighting – General,* do not permit closing of traffic lanes for routine maintenance of the lighting system on any roadway having a posted speed limit above 25 mph.

2-15 Review of Adjustment Plan

Standard Specifications, Section 48-1.01C(3), *Adjustment Plan Shop Drawings,* requires the contractor to submit an adjustment plan if the falsework needs to be adjusted more than 1/2-inch. The adjustment plan and calculations must be sealed and signed by a civil engineer registered in the State of California.

The adjustment plan may be designed and submitted as part of the falsework shop drawing submittal or designed and submitted as a separate action submittal, which is reviewed and authorized pursuant to *Standard Specifications,* Section 5-1.23B, *Action Submittals*. If the adjustment plan is part of the falsework shop drawing submittal, authorization of the shop drawings will constitute authorization of the adjustment plan as well.

The adjustment plan must be authorized by the engineer prior to any adjustment of the falsework. The review of the adjustment plan must be in accordance with the review process for shop drawings in this chapter.

Chapter 3: Loads

Table of Contents

3-1 Introduction

Falsework must be designed to resist the sum of all dead and live vertical loads, plus an assumed horizontal load, as provided in the *Standard Specifications*, Section 48-2.02B(2), *Falsework – Design Criteria – Loads*.

The vertical loads include:

- Dead load, which includes weight of concrete, reinforcing steel, forms, and falsework.
- Live load, which includes equipment, crew, and tools.
- Minimum load, which is applied to provide an acceptable strength of all falsework members.

The assumed horizontal loads include:

- A sum of equipment, construction sequence, other causes, and wind loading.
- A minimum horizontal load of 2% of the total weight of the superstructure and falsework applied during the unloaded and loaded stage of the falsework.

Loads due to differential settlement must also be considered in the design. Modified vertical design loads and traffic impact loads are applied to falsework over or adjacent to roadways and railroads.

Due to the temporary nature of falsework, earthquake loads are not considered. The probability of an earthquake occurring while the falsework is up is very low. However, there is some probability of an earthquake occurring during stage construction. Therefore, the bridge designer is directed to consider a reduced earthquake loading on partially completed structures over or adjacent to traffic as stated in *Memo to Designers (MTD)*, 20-2, *Site Seismicity for Temporary Bridges and Stage Construction*.

3-2 Vertical Load

3-2.01 Dead Load

When calculating the dead load imposed on a falsework member (except for deflection as discussed below) the dead load imposed is the weight of the:

- Concrete
- Forms
- Reinforcing steel

- Self-weight of the member

The minimum value given in the Standard Specifications, Section 48-2.02B(2), Falsework – Design Criteria – Loads, for the weight of the concrete, forms, and reinforcing steel is:

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

For typical concrete bridges the weight of forms and rebar may be estimated as:

- 15 pcf

When calculating deflection as allowed by the *Standard Specifications*, Section 48-2.02B(3), *Stresses, Loadings, and Deflections*, the dead load on the member is the weight of the reinforced concrete only (see Section 4-2.01, *Beam Deflection*). For the dead load calculation, it is customary to use:

- 150 pcf for normal concrete.
- The actual value as determined from unit-weight tests for lightweight concrete.

Falsework must be designed to support the dead load of the entire superstructure cross section, excluding the weight of the bridge railings during the unloaded and loaded stages.

There is an exception for box girder stems and soffit. Girder stems may be considered self-supporting between falsework bents if the following conditions are met:

- Distance between falsework bents does not exceed 4 times the depth of the portion of the girder stem placed in the first pour
- Deck concrete is placed more than 5 days after girder stem concrete

This exception is based on strut-and-tie modeling. The purpose of this exception is to reduce the design dead load on joists and stringers for box girder bridges in those cases where the girder stems and soffit have gained sufficient strength to carry the weight of the deck.

3-2.02 Live Load

The design live load consists of a combination of:

- 20 psf uniform load applied over the total area supported.
- The actual weight of construction equipment applied as a concentrated load at each point of contact.

- 75 pounds per lineal foot (plf) uniform load applied at the outside edge of deck overhangs.

Engineering judgment is required when investigating the effect of live loads caused by construction equipment. Some instances will occur where equipment live load and concrete dead load are not applied at the same time.

For application of the uniform 20 psf live load, the total area supported includes the area of construction walkways that extend beyond the outside edge of the deck or the deck overhang. However, the design load for all falsework supporting the walkway is the greater of the actual vertical load, or the minimum total design load of 100 psf, as discussed in the following section.

3-2.03 Minimum Total Design Load

3-2.03A Introduction

The *Standard Specifications,* Section 48-2.02B(2), *Falsework – Design Criteria – Loads*, require that the minimum total design load, dead load plus vertical live load, to be used in the design of any member must not be less than 100 psf.

The 100 psf load represents a combination of dead and live loads including miscellaneous loads such as crew, tools, equipment, and material staging. This load is in line with Cal-OSHA requirements for falsework.

3-2.03B Application

For application of this requirement, the meaning of the term “total area supported” includes any area that is subjected to dead load and/or live load during any construction sequence.

Referring to Figure 3-1, *Walkway Support Members*, the overhang joist, header, post, soffit joist, stringer, and members supporting the stringer, all see the construction walkway area as part of the “total area supported.” See also Figure 3-2, *Edge of Deck and Walkway Loading*.

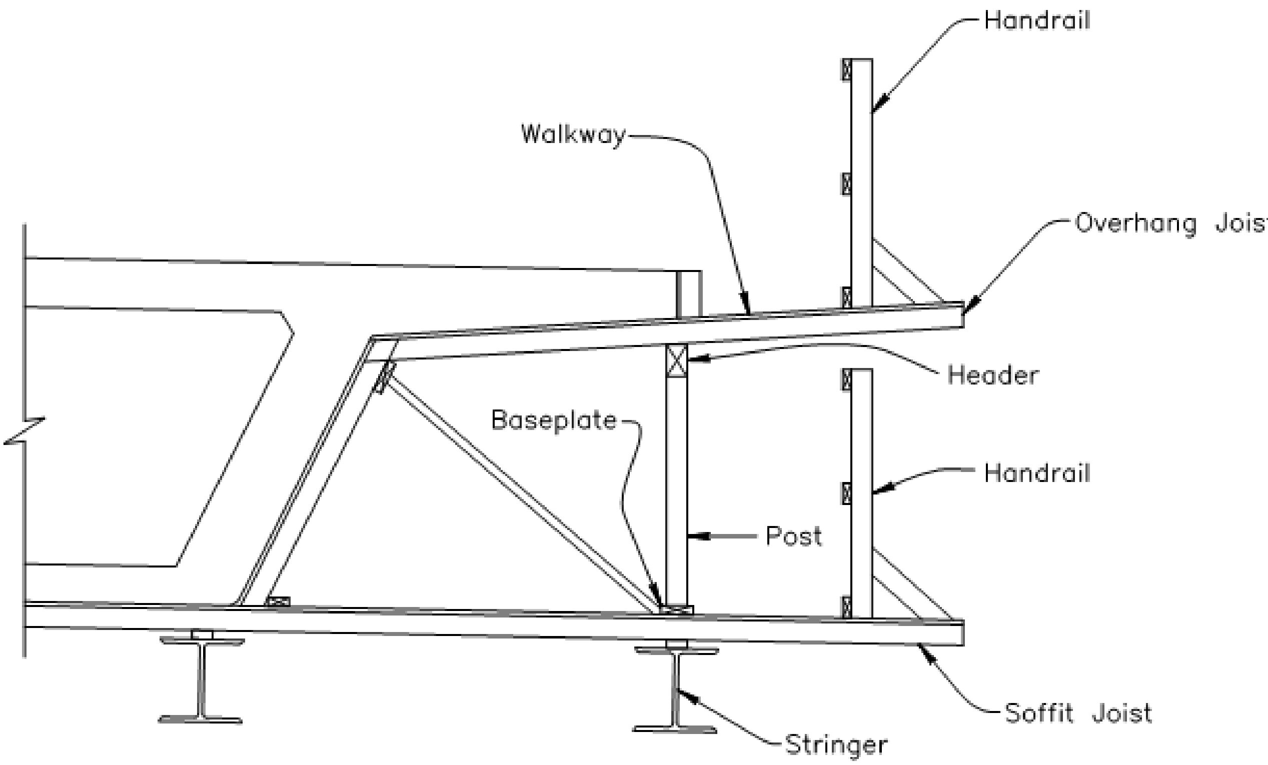


Figure 3-1. Walkway Support Members.

3-2.04 Deck Overhangs

3-2.04A Introduction

Experience has shown that concentrated live loads, such as the load from working, finishing, and curing the concrete and other miscellaneous small equipment and materials not otherwise considered, can and do occur at or near the edge of a bridge deck during the concrete placing and finishing operations. In the case of deck overhangs, these loads may significantly increase the stresses in the overhang falsework support system.

The contractor may use a variety of equipment to construct bridges and place, finish, and, cure concrete, such as belt spreaders, concrete pavers (Bidwell), and concrete buggies. See also Section 3-2.04D, *Loaded Zone,* for miscellaneous equipment and material not otherwise considered, which are used during concrete placement and finishing operations.

3-2.04B Application

Referring to Figure 3-2, *Edge of Deck and Walkway Loading,* to account for the accumulated effect of the loads mentioned above, the *Standard Specifications,* Section 48-2.02B(2), *Falsework – Design Criteria – Loads,* include the requirement of:

- Dead load.

- 75 plf live load applied along the outside edge of all deck overhangs, applied over 20 feet, see Section 3-2.04D, *Loaded Zone*. This load represents the concrete finishing and curing operations and other miscellaneous small equipment and materials not otherwise considered.
- Concentrated equipment load from concrete bridge pavers, etc.
- 20 psf uniform live load applied over the total area supported by the falsework.
- 100 psf minimum total design load (also applied on construction walkway adjacent to the edge of the deck overhang).

The uniform load of 75 plf is only applied at the edge of deck overhangs. It is not applied along the edge of slab bridges or box girder bridges without overhangs.

Figure 3-2, *Edge of Deck and Walkway Loading*, is a schematic of the various loads and load combinations specified for design of the deck overhang falsework.

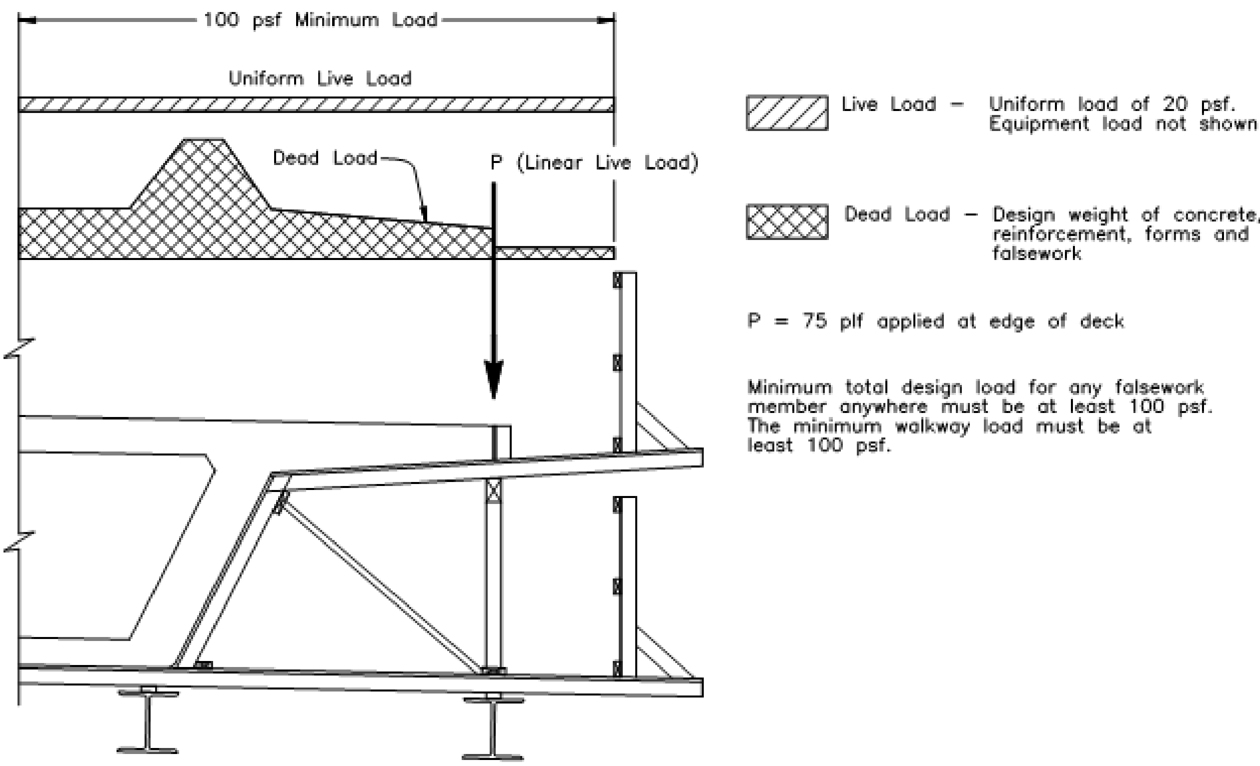


Figure 3-2. Edge of Deck and Walkway Loading.

3-2.04C Deck Overhang Brackets

For deck overhang brackets, the 75 plf and the Bidwell wheel load should not be added together but be considered separately. Use the controlling load for the design of the overhang bracket. The reasoning is that the 75 plf is more likely to occur in front of or behind the Bidwell rather than beside it, hence the individual overhang brackets will only see one of these loads at any given time.

3-2.04D Loaded Zone

While the 75 plf load is a necessary design consideration for deck overhang falsework, its application to falsework components below the overhang support system may, in the case of long falsework spans, impose a design load that is unlikely to occur in actual practice. To prevent an unrealistic loading condition for falsework members, the distance over which this load is applied is limited to a loaded zone of 20 feet in length, measured along the edge of the overhang. The loaded zone will be viewed as a moving load positioned to produce maximum stresses in the member under consideration.

The loaded zone concept may be used when checking stresses in stringers, caps, posts, and other members of the falsework system, below the level of the bridge soffit, in all cases where the falsework spans exceed 20 feet in length.

This loaded zone concept will be applied to the following two cases:

- Application of the 75 lbs/ft live load on the edge of deck
- The minimum total design vertical load (100 psf) on a construction walkway adjacent to the edge of the deck overhang

3-2.05 Falsework Over or Adjacent to Roadways or Railroads

3-2.05A Introduction

The *Standard Specifications,* Section 48-2.02B(4), *Design Criteria – Special Locations,* include specific requirements for falsework over or adjacent to roadways and railroads. For a more detailed explanation of these requirements, see Section 4-12, *Falsework Over or Adjacent to Roadways or Railroads*.

3-2.05B Modified Design Load

The vertical design load for posts and towers, over or adjacent to roadways and railroads, must be designed for the greater of:

- 150% of the calculated post load, not including any increased or readjusted loads caused by prestressing.
- Increased or readjusted loads caused by prestressing.

The modified design load also applies to posts and towers that are adjacent to roadways and railroads, which do not support falsework members over traffic, but are within the limits shown in Section 4-12.01, *Introduction*. For more details, see Section 4-12.05E, *Modified Design Load*.

3-3 Horizontal Load

3-3.01 Introduction

The falsework bracing system must be capable of resisting an assumed horizontal load applied in any direction. The specified horizontal design load is an assumed load. Since it is an assumed load, it will not necessarily be equal to any actual horizontal load that may occur. Nevertheless, the bracing system must be designed to resist the assumed horizontal load to ensure stability at all stages.

The minimum assumed horizontal load will generally govern the design for typical highway bridges and other structures where the falsework height is less than 30 feet. Depending on configuration, wind loads may govern when the height of falsework exceeds 30 feet and wind loads will govern most designs where height exceeds about 40 feet.

3-3.02 Application

For typical analysis, the horizontal load is applied at the top of the post (bottom of top cap).

The design horizontal load:

- Is the sum of any actual loads due to equipment, construction sequence or other causes, plus the wind load?
- The assumed horizontal load must not be less than 2% of the total dead load at the location under consideration. The total dead load includes the weight of the falsework to be supported and the total weight of the new structure to be supported.

The falsework bracing system must be designed to resist the assumed horizontal load with the falsework in both the:

- Unloaded condition.
- Loaded condition.

For concrete structures the weight of forms and rebar may be used to resist the overturning in the unloaded condition. See Section 6-5.04, *Resisting Moments.*

The falsework bracing must be designed to resist the assumed horizontal load to ensure both transverse and longitudinal stability. Falsework system stability is discussed in Chapter 6, *Stability*.

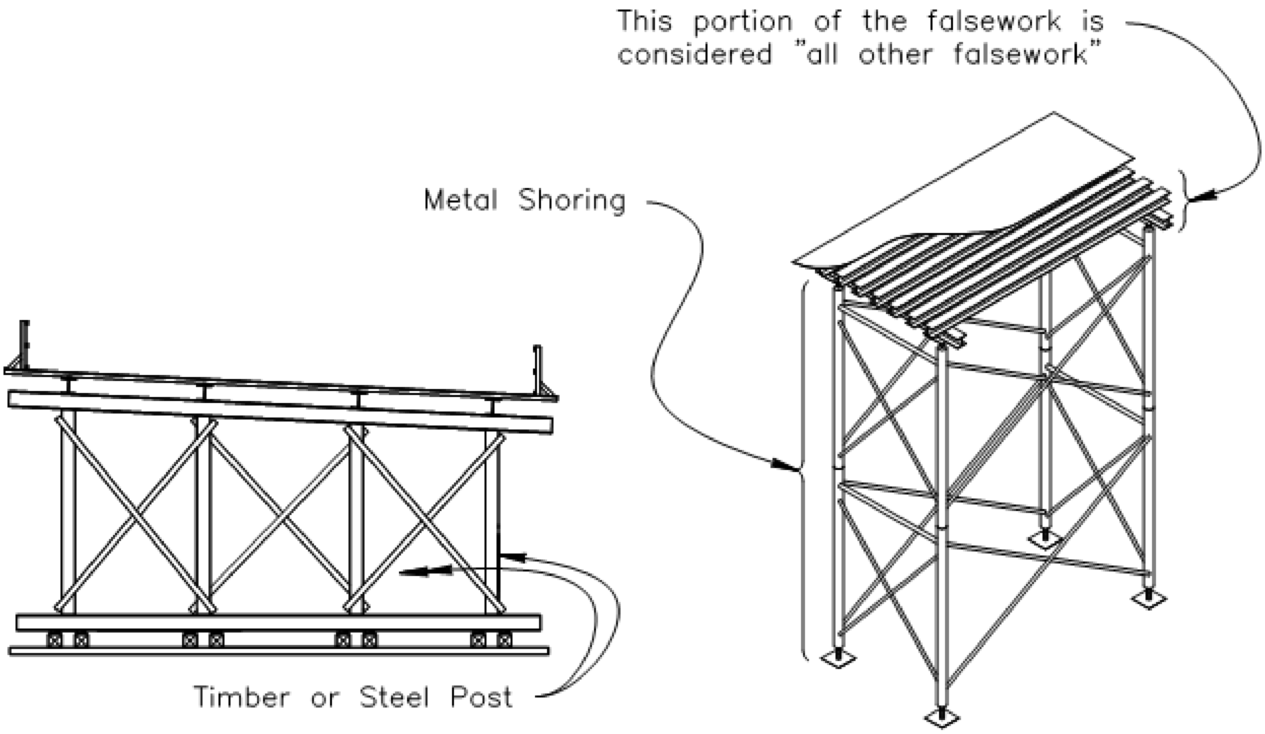
3-3.03 Wind Loads

Determining the actual force exerted by wind on bridge falsework is a highly indeterminate problem due to the number of variable factors involved. Although, it is possible to establish values for all the variables in a falsework system, it is quite cumbersome and time consuming. More complex methods of calculation are often no more accurate than a simplified method because of the subjective nature of some variables. Therefore, SC has developed *Standard Specification*s, Section 48-2.02B(2), *Falsework – Design Criteria – Loads*, which recognizes the effect of the more influential variables and assigns a coefficient to cover the others. By using this simplified method, there is statewide uniformity in calculating wind loads.

The wind pressure values were developed from the basic theory of fluid flow with a design wind velocity based on a 100-year recurrent wind prevalent in most of California. Reductions were taken to account for the temporary exposure of falsework to wind forces and increased for gusts. Variables are wind velocity, downwind width of the system, downwind distance between adjacent members, drag or shape factor for the various members, “solidity ratio” or percentage of solid-surface in a given gross frontal area, and height of the falsework above the ground.

For the wind load calculation, the specification considers two general falsework types:

- Heavy-duty metal shoring and steel pipe column falsework with a vertical load carrying capacity greater than 30 kips per tower leg or pipe column.
- All other falsework, which includes timber post, metal pipe frame, and metal shoring systems. It also includes falsework above the heavy-duty shoring or pipe columns. For some examples, see Figure 3-3, *Examples of Falsework for Wind Loading.*



a) All Other Falsework **b) Heavy Duty Metal Shoring**

Figure 3-3. Examples of Falsework for Wind Loading.

For heavy duty shoring and pipe column falsework systems, the wind load is the product of the wind impact area, a shape factor, and an appropriate wind pressure value. The wind impact area is defined as the total projected area of all elements in a tower face or bent normal to the direction of the wind. The shape factor is included to account for the effect of wind drag forces on the members and, for heavy duty shoring, the effect of wind acting on members in the other three tower faces. See Section 3-3.03A, *Wind Load on Heavy Duty Metal Shoring,* for more details.

For all other falsework, including falsework supported by heavy duty shoring and pipe columns, the wind load is the product of the wind impact area and an appropriate wind pressure value. The wind impact area is the gross projected area of the falsework and any unrestrained portion of the permanent structure, e.g. bridge precast girder or steel girder without deck slab, excluding the area between falsework bents or towers where diagonal bracing is not used. In the specification context, the term "diagonal bracing" does not include flexible bracing systems, such as cable, reinforcing steel bars, steel rods and bars, and similar members that do not resist compression.

For all falsework types, the wind pressure value is a function of the height of the falsework. Wind pressure values, for each height zone, are tabulated in the *Standard Specifications,* Section 48-2.02B(2), *Falsework – Design Criteria – Loads*.

Wind pressure height zones are always measured from the ground up regardless of falsework configuration. Except for falsework on driven pile bents, the height to be used for the wind impact area calculation is the vertical distance between the base elevation of the component of the system, about which overturning rotation can occur, and the bridge soffit. In the case of pile bents, judgment is required to determine the lower limit of the wind impact area. If the piles are cut off and capped near the ground, the lower limit will be the plane at the pile cut-off elevation. If, however, the piles extend an appreciable distance above the ground or above the water surface for structures over water, the entire height of the falsework (measured from ground or water surface to bridge soffit) should be used.

The formwork extending above the bridge soffit should not be included in the wind impact area. This formwork is excluded from the wind impact area under the assumption that when subjected to the design wind load, the forms would be blown off the falsework. However, the contractor should restrain their forms on top of the falsework in the event of high winds.

Example problems illustrating the procedure to be followed when calculating the wind load on various falsework systems are included in Appendix D, *Example Problems*.

3-3.03A Wind Load on Heavy Duty Metal Shoring

For wind acting on heavy-duty steel shoring with a vertical capacity of more than 30 kips per leg, the critical loading condition will occur when the wind force is applied at right angles to the tower faces. The effect of wind acting in other directions need not be considered except in the case of temporary bracing installed during erection and/or removal. See Figure 3-4, *Tower Leg Configurations,* for towers with various leg configurations.

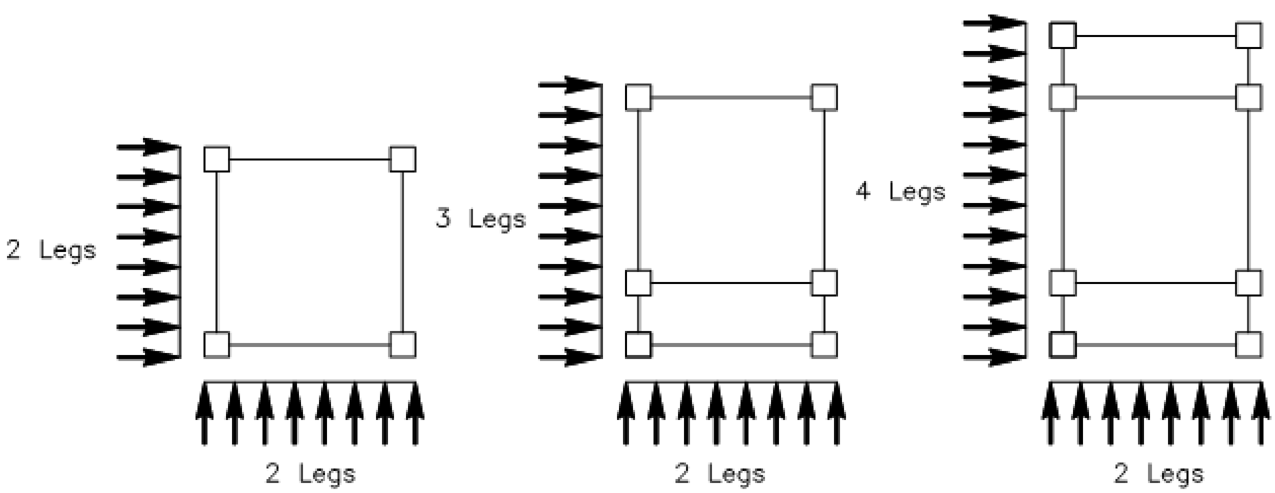


Figure 3-4. Tower Leg Configurations.

Referring to Figure 3-5, *Wind Load*, the horizontal design load produced by wind forces acting on top of the heavy-duty steel shoring is determined as follows:

1. From the table in the *Standard Specifications,* Section 48-2.02B(2), *Falsework – Design Criteria – Loads*, select the wind pressure for each height zone.
2. Multiply the selected wind pressure by the specified shape factor of 2.2 to obtain the design wind pressure.
3. Calculate the total wind force per tower, **WF**, for each height zone by multiplying the design wind pressure by the total projected area of all the elements in the tower face normal to the applied wind.
4. For each height zone, calculate the overturning moment by multiplying the wind force by the distance, **h**, from the base of the tower (top of the tower footing) to the center of pressure. Add the overturning moments for each height zone to obtain the total overturning moment.
5. Divide the total overturning moment by the vertical distance between the tower base and a horizontal plane at the top of the highest tower. The value thus obtained is the horizontal design wind load, **DWL**, acting on top of the tower.

This method calculates the wind load to be applied at top of the tower, so it can easily be compared to the assumed horizontal load, which is also applied at the top of the tower. The greater of the two loads is used for design, see also Example 5, *Wind Load on Heavy Duty Falsework*, in Appendix D, *Example Problems*.

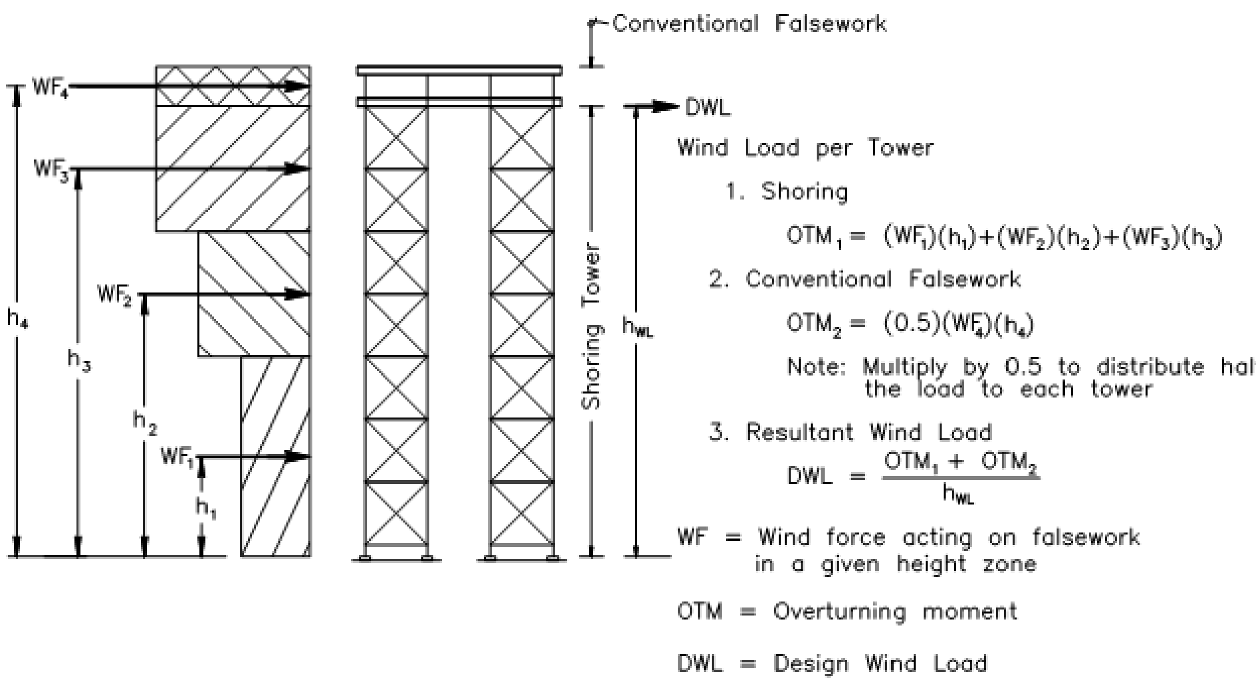


Figure 3-5. Wind Load.

3-3.03A(1) Analysis in the Transverse Direction

Except as provided in the following paragraph, adjacent towers in the same falsework bent must each resist the design wind load because the upwind tower does not shield the downwind tower to any significant degree. This premise will be considered valid regardless of the distance between the towers and will include those configurations where the space between abutting legs of adjacent towers is minimal. See Figure 3-6, *Wind Load on Independent Abutting Towers*.

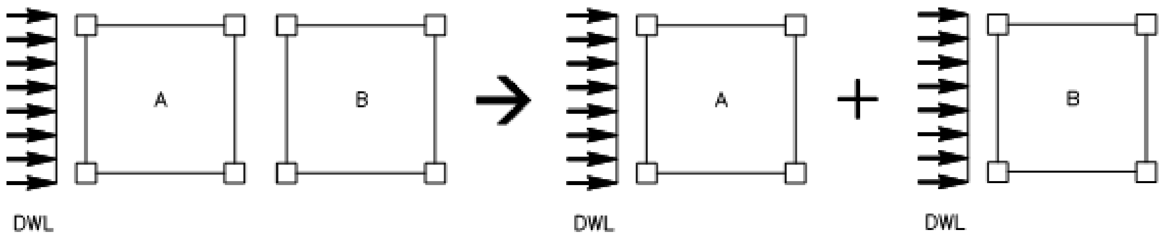


Figure 3-6. Wind Load on Independent Abutting Towers.

If the abutting legs of adjacent towers are connected, the total wind load for the two towers will be assumed as 1.5 times the design wind load acting on the upwind tower face. For analysis of the system, distribute 1/2 of the total wind load (or 75% of the design wind load) to each tower. See Figure 3-7, *Wind Load on Connected Abutting Towers*.

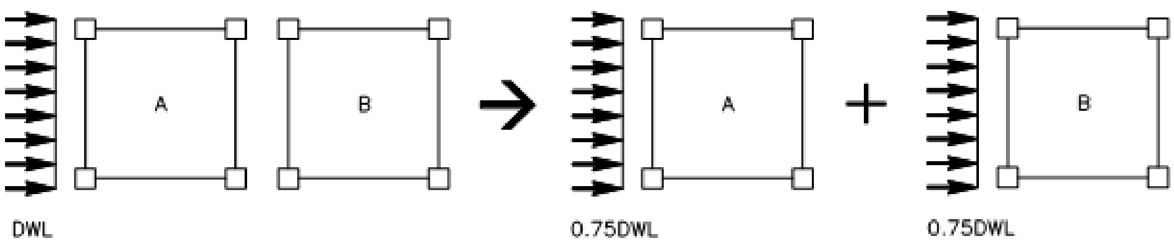


Figure 3-7. Wind Load on Connected Abutting Towers.

In addition to resisting the horizontal load produced by wind acting on the shoring towers, the bracing system must resist the additional horizontal load produced by wind acting on elements of the system supported by the shoring (caps, stringers, joists, etc.). The design wind load on supported falsework is calculated as wind load on “all other falsework”. See Section 3-3.03C, *Wind Load on All Other Falsework*.

Refer to Figure 3-8, *Wind Load on Tower from Supported Falsework,* for wind acting parallel to the falsework bent, the wind load on the supported falsework will be distributed to the individual towers in accordance with the following assumptions:

- For bents with two towers, 1/2 of the design wind load will be distributed to each tower.
- For bents with three towers or more, 1/2 of the design wind load will be distributed to the upwind tower and the remainder distributed equally to all other towers in the bent.

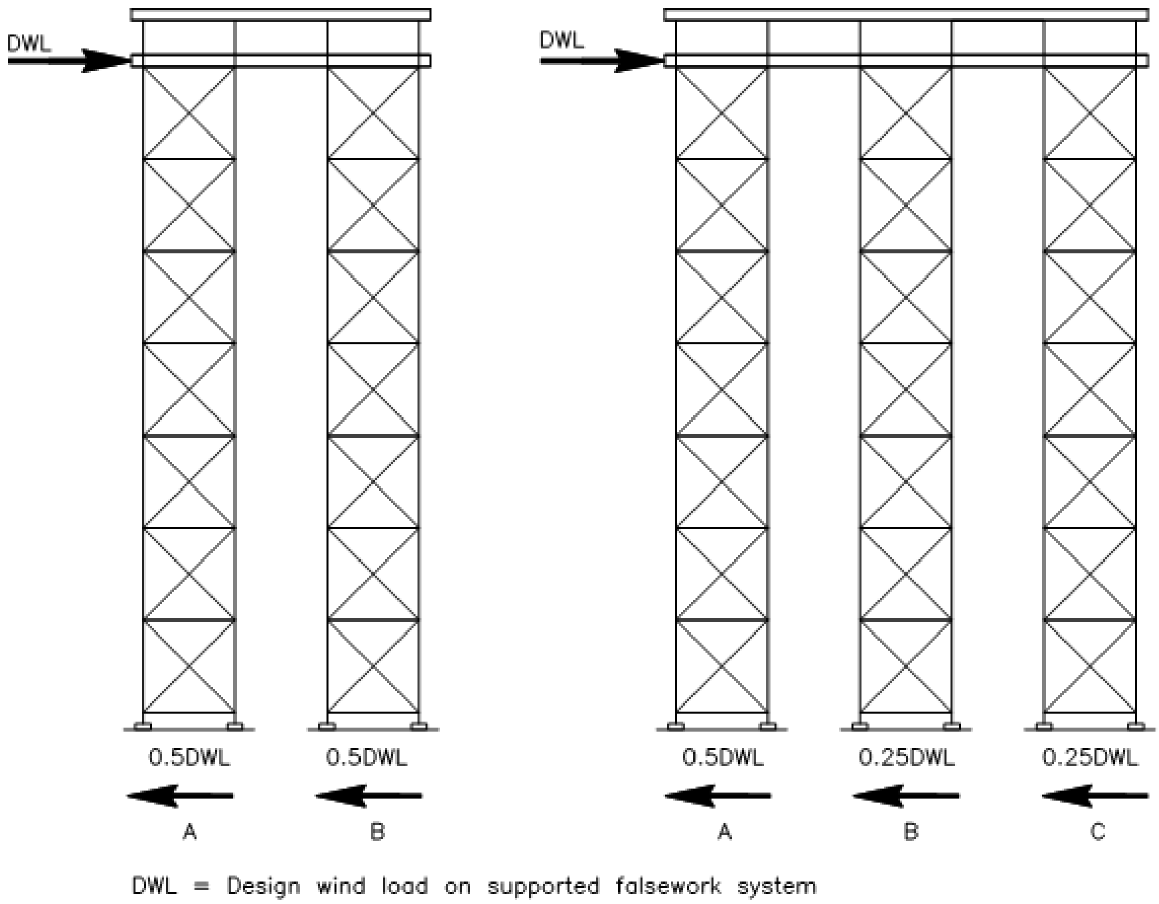


Figure 3-8. Wind Load on Tower from Supported Falsework.

3-3.03A(2) Analysis in the Longitudinal Direction

For wind acting in the longitudinal direction or normal to the bent, the overturning moment calculation (wind load per tower) will be as depicted in Figure 3-5, *Wind Load,* for wind acting on the towers. However, distribution of the load produced by wind acting on the supported falsework depends on the way the system is designed to resist longitudinal forces. The load due to wind acting on the supported falsework should be distributed to the system in accordance with the discussion in Section 6-4, *Longitudinal Stability*.

3-3.03B Wind Load on Pipe Column Falsework

For a pipe column falsework bent with a capacity greater than 30 kips per pipe column, the horizontal design load due to wind acting on the bent is the sum of the wind loads on the individual pipe columns in the bent. While this is obvious for wind acting normal to the bent, it is also the case for wind acting in the transverse direction (parallel to the bent centerline). Typically, the columns are so widely spaced that shielding will not occur.

For adjacent columns where the downwind column is within the shielded zone, the applied wind force on the downwind column will decrease. This will be offset by higher drag forces produced by increased wind turbulence. Because of this offset, ignore any theoretical decrease in wind load attributable to downwind shielding of adjacent pipe columns. See Section 3-3.03D, *Effect of Shielding on Wind Impact Area,* for a discussion of shielding of downwind falsework members.

The shape factor of 1.0 for pipe column falsework specified in the *Standard Specifications,* 48-2.02B(2), *Loads*, has been adjusted upward from the common shape factor of 0.8 for circular sections, to account for the effect of bracing and connections, which are ignored in the calculations. This procedure is reasonable for bents with more than 30 kips per pipe column where the bracing consists of small diameter steel rods or reinforcing steel, cable, or small structural shapes. However, in the event larger bracing elements are used, the projected area of the bracing components must be included in the total projected area of the falsework calculated in step 3. For this calculation, use a shape factor of 1.3.

The design wind load is determined as follows:

1. From the table in the *Standard Specifications,* Section 48-2.02B(2), *Falsework – Design Criteria – Loads*, select the wind pressure for each height zone.
2. For each height zone, multiply the selected wind pressure by the specified shape factor of 1.0 to obtain the design wind pressure.
3. For each height zone, calculate the total projected area of the falsework bent. The total projected area is the sum of the projected areas (height of pipe column multiplied by the diameter) of the individual pipe columns in the bent.
4. For each height zone, multiply the design wind pressure from step 2 by the total projected area to obtain the wind force.
5. For each height zone, calculate the overturning moment by multiplying the wind force by the vertical distance between the point at the base of the pipe column frame about which overturning rotation will occur and the center of wind pressure.

6. Add the overturning moments for each height zone to obtain the total overturning moment.
7. Divide the total overturning moment by the vertical distance between the point of overturning rotation at the base of the frame and the top of the highest bent component. The value thus obtained is the horizontal design load for wind acting on the bent.

Engineering judgment is required when determining the point about which overturning rotation will occur. Typically, a pipe column bent is a rigid unit consisting of top and bottom cap beams, two or more columns, and internal diagonal bracing, all supported by a foundation system. Where vertical/grade adjustment is provided at the top of the bent, the lower cap will be supported by corbels, which distribute the load to the foundation. Where vertical/grade adjustment is provided at the bottom, wedges will be located between the bottom cap and the corbels. In either of these typical cases, when overturning forces are applied, the bent will tend to rotate about a point at the bottom of the lower cap beam. For other configurations, the point of rotation should be determined as the lowest point in the system about which rotation can occur while the frame remains rigid.

In addition to resisting the horizontal load produced by wind acting on the falsework members in the bent, the bracing must resist the additional horizontal load produced by wind acting on elements of the falsework supported by the columns (caps, stringers, joists, etc.). The design wind load on supported falsework is calculated as wind load on “all other falsework”. See Section 3-3.03C, *Wind Load on All Other Falsework*.

3-3.03C Wind Load on All Other Falsework

The design wind load to be applied to all other falsework, including falsework supported by heavy duty shoring and pipe column bents, is the product of an appropriate wind pressure value and the wind impact area of the system under consideration.

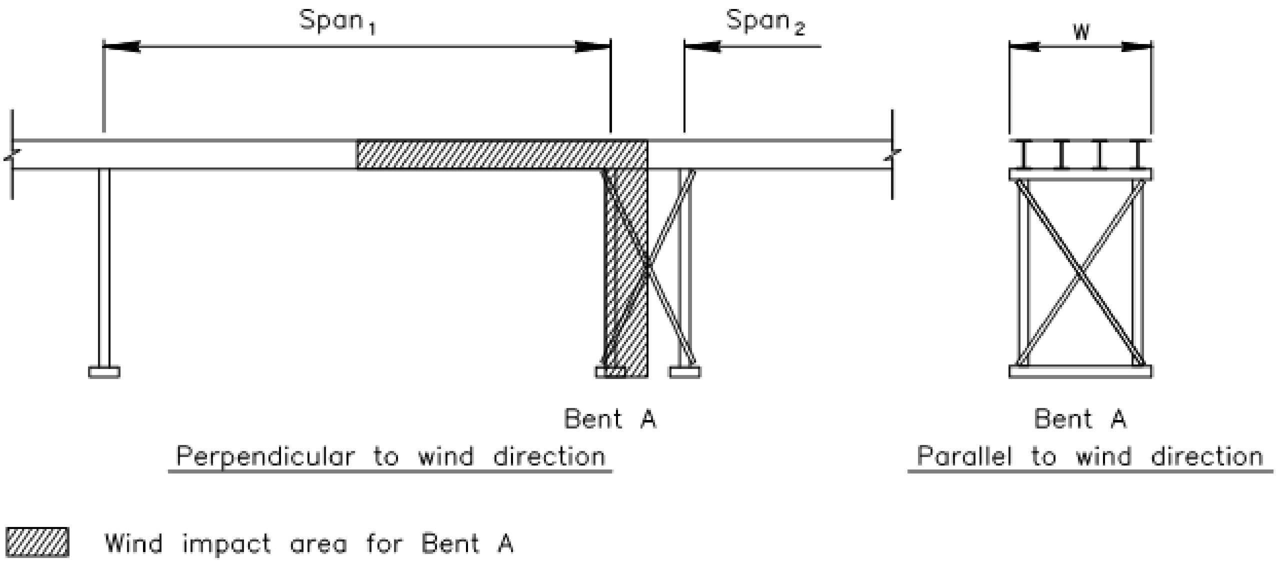


Figure 3-9. Wind Impact Area on All Other Falsework.

The design wind load is calculated as follows:

1. Determine the value for **W**, which is the downwind width of the falsework system, or that portion of the system under consideration, measured in the wind direction. For falsework supported by heavy duty shoring or pipe column bents, **W** will be the distance between the exterior beams or stringers. For all other falsework, **W** is the width of that portion of the falsework which supports a continuous cap or is connected by uninterrupted diagonal bracing.
2. Calculate the value for drag coefficient, **Q**. From the *Standard Specifications,* Section 48-2.02B(2), *Falsework – Design Criteria – Loads*:

$$\mathbf{Q = 1.0 + 0.2W \ \leq 10} \qquad (3\text{-}3.03C\text{-}1)$$

   where **Q** = Drag coefficient (psf)

   W = Width of the falsework system as determined in step 1 (ft)

   Calculate the wind pressure value for each height zone. Use the wind velocity coefficient for that height zone as listed in the table in the *Standard Specifications,* Section 48-2.02B(2), *Falsework – Design Criteria – Loads*, and the value for **Q** calculated in step 2.

3. Calculate the wind impact area, refer to Figure 3-9, *Wind Impact Area for All Other Falsework*. It is defined in the specifications as the gross projected area of the falsework and any unrestrained element of the permanent structure, excluding the area between falsework bents where diagonal bracing is not used. The term “diagonal bracing” as used in the wind impact area definition does not include flexible bracing.

4. Calculate the total wind force for each height zone by multiplying the calculated wind pressure value by the wind impact area for that height zone.
5. Calculate the overturning moment for each height zone by multiplying the wind force by its distance above the point at the base of the falsework about which overturning rotation will occur. For this calculation, the wind force will be assumed as acting at the centroid of the wind impact area for the height zone under consideration.
6. Add the overturning moments for each height zone to obtain the total overturning moment.
7. Divide the total overturning moment by the distance from the point at the base of the falsework about which overturning rotation will occur to the top of the falsework post (bottom of top cap). The value thus obtained is the horizontal design load for wind acting on the system.

Overturning failure occurs when a rigid element of the system, such as a braced frame or tower, rotates about the lowest downwind point of frame or tower support. Depending on how the post or leg loads are distributed to the foundation, the point of overturning rotation might be at the top of a corbel or other load distributing member rather than at the bottom of the system as a whole.

For typical falsework, the wind force should be applied parallel to and perpendicular to the longitudinal axis of the falsework bent. The effect of wind acting in other directions typically does not need to be considered in the analysis.

For wind forces (or a wind force component) applied parallel to the axis of a falsework bent, the calculated design wind load for each width, **W**, must be resisted by bracing within that width.

For wind forces applied perpendicular to the bent, resistance to the design wind load should be evaluated in the same manner as resistance to other longitudinal forces.

3-3.03D Effect of Shielding on Wind Impact Area

When investigating the effect of wind acting perpendicular to a falsework bent, consideration may be given to the shielding provided by solid obstructions. Solid obstructions such as abutment fills and pier walls will shield downwind falsework members to some extent, and thus reduce the wind impact area. The degree of shielding provided is not easily determined. To ensure uniformity, apply the assumptions discussed in the following paragraphs.

As wind blows around the end of a solid obstruction, the area over which the wind pressure is effective will increase inward on a 2:1 ratio (downwind distance to inward distance) as shown in Figure 3-10, *Wind Shielded Zone Limits*. Falsework bents within the shielded zone will be considered as totally sheltered from wind forces.

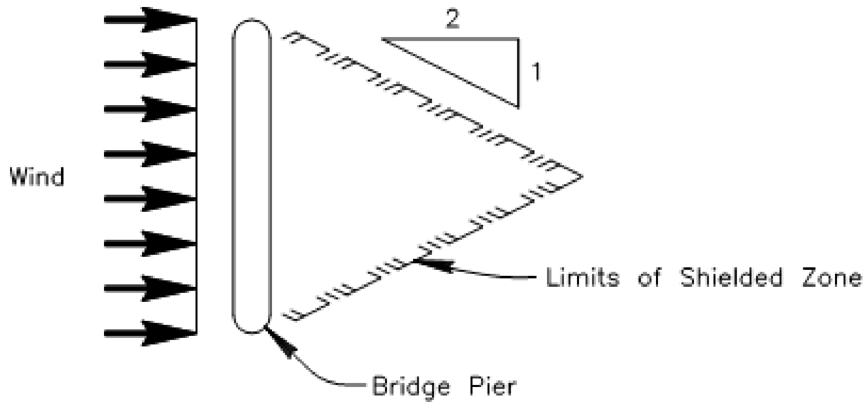


Figure 3-10. Wind Shielded Zone Limits.

Referring to Figure 3-11, *Wind Load on Continuous and Discontinuous Bents*, when bents are located immediately adjacent to solid obstructions and are almost fully shielded, the effect of wind load may be neglected, see Bents 1 and 6. When falsework bents are partially shielded, the term “gross projected area of the falsework” will be interpreted as the area of the bent that is outside the shielded zone, see Bents 2 through 5. For discontinuous bents, such as Bent 2, which is partially shielded, the wind load on Bent 2L may be distributed to the length of the Bent 2L. For continuous bents, such as Bent 5, which is partially shielded, the wind load on Bent 5 may be distributed to the entire length of Bent 5.

Wind may blow from any direction. Falsework bents that are totally shielded from wind in one direction may be fully exposed when the wind forces are applied from the opposite direction.

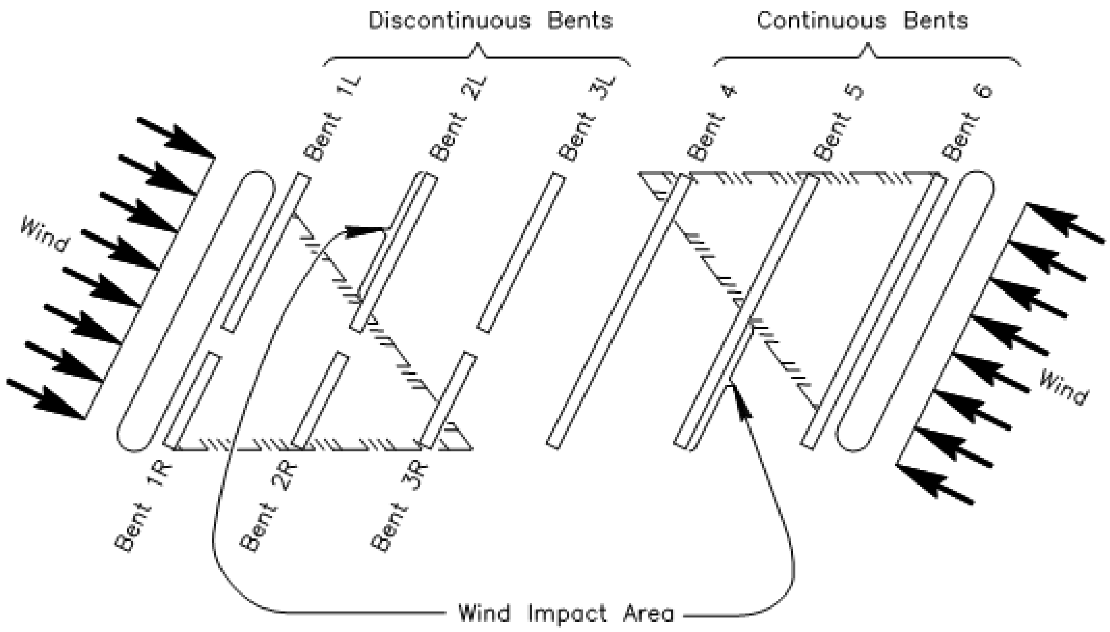


Figure 3-11. Wind Load on Continuous and Discontinuous Bents.

3-3.04 Falsework Over or Adjacent to Roadways or Railroads

The *Standard Specifications,* Section 48-2.02B(4), *Design Criteria – Special Locations,* include specific requirements for falsework over or adjacent to roadways and railroad. For a more detailed explanation of these requirements, see Section 4-12, *Falsework Over or Adjacent to Roadways or Railroads*.

Impact loads are required to be applied at various locations in falsework adjacent to or over roadways and railroads and within the limits described in Section 4-12.01, *Introduction*. For a more detailed explanation of the impact load and the required connections, see Section 4-12, *Falsework Over or Adjacent to Roadways and Railroads*, Section 4-12.05E, *Modified Design Load*, and Section 4-12.06, *Additional Requirement Over or Adjacent to Railroads*.

3-3.05 Stream Flow

When falsework supports are placed in flowing water, water pressure on the supports are determined by the following formula:

$$P_w = Kv^2 \qquad (3\text{-}3.05\text{-}1)$$

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

Chapter 4: Design Considerations

Table of Contents

4-1 Introduction

This chapter covers design considerations which must be addressed during design and review of shop drawings. Subsequent chapters cover specific design methods and procedures.

Shop drawings must not be authorized if the applicable design considerations have not been addressed properly in the design.

4-2 Deflection and Camber

4-2.01 Maximum Allowable Deflection

The maximum allowable beam deflection is limited to:

$$\Delta_{\mathbf{max}} \leq \frac{\mathbf{L}}{\mathbf{240}} \tag{4-2.01-1}$$

where $\Delta_{\mathbf{max}}$ = Max allowable beam deflection

$\mathbf{L}$ = Span length of falsework beam

The deflection is calculated using the weight of all the concrete in the whole superstructure cross section, as though the entire superstructure were placed in a single concrete pour; the weight of the falsework is not included in the calculation. See *Standard Specifications*, Section 48-2.02B(3), *Stresses, Loadings, and Deflections*. This limiting value is included in the specifications to ensure a certain degree of rigidity in the falsework and thereby minimize distortion of the forms as concrete is placed.

4-2.02 Actual Deflection

The actual deflection is the deflection that occurs as the falsework beam is loaded. Calculating actual deflection is the engineer's responsibility, since it is used in determining the amount of falsework camber required.

When calculating the actual deflection, include the weight of:

- Concrete, reinforcement, and forms (160 pounds per cubic foot (pcf)).
- The supporting beam (pounds per linear foot (plf)).

Consideration must be given to such factors as the sequence of construction and the depth of the superstructure when two or more concrete pours are involved.

The specifications do not include a limiting value for live load deflection, as they are of a transient nature. However, when a bridge deck finishing machine is supported at the outer edge of a cantilevered deck overhang, particular care must be taken to prevent excessive deflection of the deck overhang support system. Unless special precautions are taken, the concentrated load, due to the weight of the finishing machine, may cause the deck overhang to deflect appreciably with respect to the remainder of the deck surface. This will decrease bridge deck thickness and reduce reinforcing steel cover, both of which are detrimental to the completed structure.

The applicable specification is the general requirement that falsework must be designed and constructed to produce, in the finished structure, the lines and grades shown on the plans. See *Standard Specifications*, Section 48-2.01A, *Temporary Structures – Falsework – Summary*. To ensure compliance with this general requirement, add the “deflection due to the weight of a deck finishing machine” to the “deflection due to the weight of the concrete”. The sum of these two deflections should not be too large as to adversely affect the character of the finished work. This will require engineering judgment. In summary, the important point is that the weight of the finishing machine be considered, and the total deflection limited to a realistic value.

4-2.03 Negative Deflection

Depending on the concrete placing sequence, negative (upward) deflection may occur where falsework beams are continuous over a long span and a relatively short adjacent span. This condition (negative deflection at the end support) is an indication of system instability and must be considered in the falsework design. If beam uplift cannot be prevented by loading the short span first, the end of the beam must be restrained, or the span lengths must be revised. Designs where theoretical beam uplift will occur under any loading condition must not be authorized.

When falsework stringers are considerably longer than the actual falsework span, the stringer cantilever extending beyond the point of support will deflect upward as the main span is loaded. The design must include provisions to accommodate this upward deflection. The usual method is to use a sleeper (filler strip) on the main span only, which allows free movement of the stringer cantilever. The sleeper should end at the center line of the falsework top cap and should not extend into the cantilever section of the stringer. The sleeper must be thick enough to offset the theoretical uplift on the cantilever, see Figure 4-1, *Sleeper on the Falsework Stringer*.

Note: No sleeper on beam tails.

Figure 4-1. Sleeper on the Falsework Stringer

Sometimes the contractor may use the steel beam cantilever beyond the support, with wood beams wedged tight between its flanges, to close the gap at abutment and bent faces. This may be acceptable for a closure distance up to 4 feet. This detail, when applied to longer distances, can cause depression in the wet soffit concrete due to stringer tail movement when concrete is placed in the main span. This should be discouraged.

4-2.04 Camber

The term “camber” is used to describe an adjustment to the profile of a load supporting beam or stringer so the completed structure will have the lines and grades shown on the plans. In theory, the camber adjustment consists of the sum of the following factors:

- Anticipated total deflection of the falsework beam (stringer) under its own weight and the actual load imposed
- Difference between the falsework beam profile and profile grade, also called vertical curve compensation
- Difference between the falsework beam profile and ultimate superstructure deflection curve (bridge camber)
- Difference between the falsework beam profile and any permanent or residual camber to remain in the structure for its useful service life

In structures with parabolic soffits, an additional adjustment may be required to account for the difference between beam profile and soffit curvature. On parabolic soffits the vertical curve component is sometimes included with the soffit profile (4-scale) grades.

When falsework beams are relatively short, the theoretical adjustment due to vertical curve compensation, bridge camber, and desired permanent or residual camber will be small and may be neglected. As falsework spans increase, these factors become increasingly significant and must be considered along with beam deflection.

More than any other single factor, the satisfactory appearance of a completed structure will depend on the accuracy of the camber used in the falsework construction. Good judgment will be required, particularly in determining the amount of camber to be used to compensate for anticipated dead load falsework deflection, take up, and settlement.

In general, the deck weight of a conventionally reinforced box girder bridge should be omitted when calculating camber, since additional stringer deflection as the deck is placed usually is insignificant. In the case of cast-in-place prestressed construction, falsework span length may be an important consideration. In such structures, judgment will be required as to the relative stiffness of the girder stems, and whether they will resist additional deflection and by how much, as the deck is placed. Experience has shown that including 10-20% of the deck weight for deflection is a reasonable estimate for typical prestressed box girder bridges.

The engineer furnishes the amount of camber to use in constructing falsework; see *Standard Specifications,* Section 48-2.03C, *Falsework – Erection*.

4-2.04A Camber Strips

When to require camber strips is a matter of engineering judgment. Generally, camber strips are not necessary unless the total camber adjustment exceeds approximately 1/4-inch for stringers supporting the exterior girders, edge of the soffit, or deck overhang, and approximately 1/2-inch for beams at interior locations. The engineer orders the contractor to furnish camber strips. See *Standard Specifications*, Section 48-2.03C, *Falsework – Erection*.

To warrant proper design and installation, camber strips must conform to the following criteria:

- 900 psi maximum allowable compressive stress for perpendicular-to-grain loading
- 1.5-inch minimum width
- 1/8-inch maximum crushing
- Must be centered along the longitudinal centerline of the falsework beam
- Structure cross slope, allowable wood crushing, and joist deflection must be considered when determining the height of the camber strip.

- The minimum height of the camber strip must be such that the joists will not come into contact with any part of the falsework beam under any loading condition.
- Must not extend onto the unloaded portion of a trailing beam cantilever
- If the amount of camber is large, as in the case where a parabolic curved bridge soffit is supported by a long falsework beam, the camber strips should be braced or built up with wide material to avoid lateral instability. The use of laterally unsupported tall, narrow camber strips is not permitted.
- A sleeper is required when the stringer does not follow bridge cross slope and camber strip does not include allowance for cross slope, see Figure 4-2, *Camber Strip and Sleeper Requirements.*

Because camber strips are an incidental part of the system, their installation seldom receives more than cursory attention. Casual treatment of camber strip installation can result in an unforeseen and undesirable loading of the falsework beam. For example, a camber strip placed at a distance away from the centerline of a steel beam may induce torsional stresses that were not considered in the design. Undesirable torsional stresses may be induced in beams supporting falsework for structures having steep cross slopes, even if the camber strip is properly placed along the beam centerline.

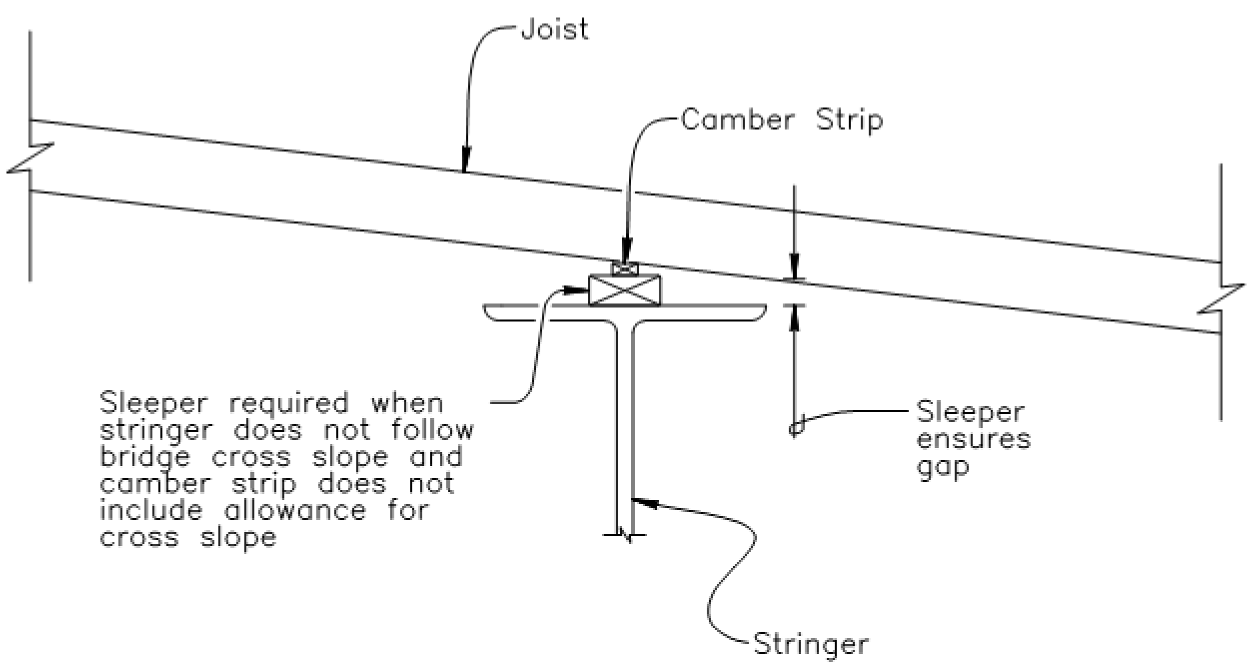


Figure 4-2. Camber Strip and Sleeper Requirements.

4-2.05 Horizontal Deflection

Although the specifications do not include a limiting value for horizontal deflection, such deflection will be negligible in any falsework system where horizontal forces are resisted by bracing. Horizontal deflection need not be considered in any case where the horizontal design load is resisted by a properly designed bracing system. This includes external bracing systems where the use of external bracing is necessary to prevent overturning.

Horizontal deflection will be a consideration when the horizontal design load is resisted by bending in a falsework member. This situation occurs when falsework is supported by pile bents. For pile bents, it is necessary to combine bending and vertical load stresses to obtain the actual stress.

The procedure for evaluating the adequacy of falsework pile bents is discussed in Section 8-6, *Pile Foundations*.

Horizontal deflection may be an issue with falsework where the stringers are loaded on the bottom flange and not directly over the web. Loading stringers this way may cause the top flange to move horizontally. See Figure 4-3, *Bottom Flange Loading*.

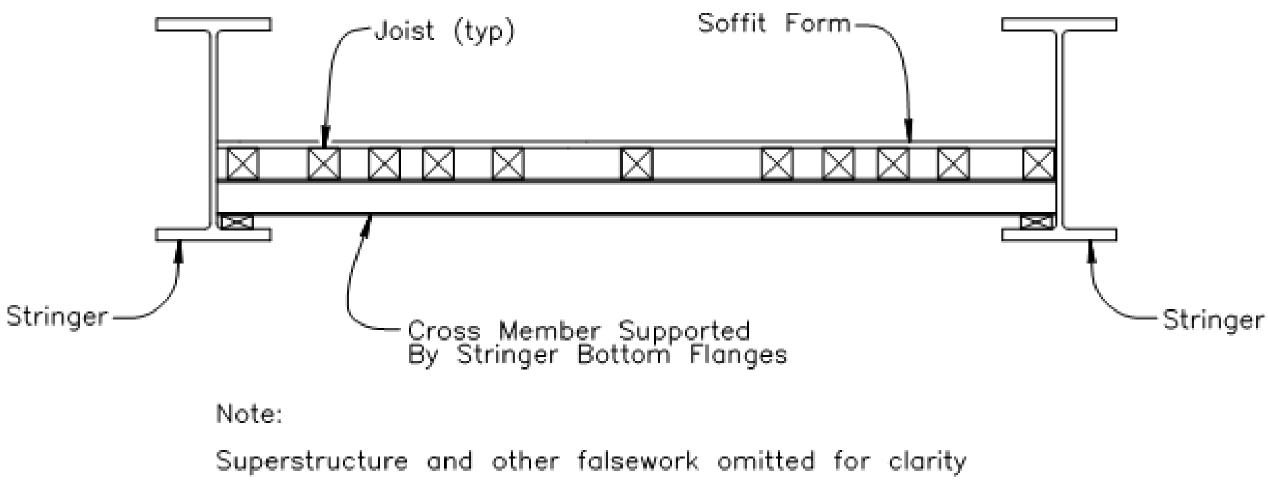


Figure 4-3. Bottom Flange Loading.

4-3 Beam Continuity

Because of the sequential, and sometimes unpredictable way falsework loads are applied, beam continuity is an uncertain design condition. To accommodate this uncertainty, assume the continuous beam condition when continuity will act to increase loads or stresses, but not otherwise.

For example, the simple span condition will be assumed when calculating positive bending moments in joists, stringers, and similar continuous members; however, full continuity will be assumed when calculating negative bending moments in these same members. Assume full continuity when calculating the beam reaction on interior supports under continuous falsework members but assume the simple span condition when calculating the reaction at the end support.

In a framed bent, continuity must be considered in any case where stringer loads are applied within the cap span rather than directly over the supporting post to ensure that allowable post loads are not exceeded.

Continuous caps are often supported by two or more towers in a heavy-duty shoring system. If leg loads are unequal, the resulting differential leg shortening will cause a redistribution of beam reactions and a corresponding change in the magnitude and location of maximum cap bending stress.

When beams are continuous over two or more spans, beam uplift can occur in adjacent unloaded spans when concrete is placed in one span. Refer to discussion in Section 4-2.01A, *Negative Deflection*.

The engineer will be expected to recognize these and other cases where the effect of beam continuity must be investigated to prevent the overstressing of any falsework member or instability in the system.

4-4 Cap Beam Center Loading Strips

4-4.01 Introduction

Timber center loading strips or shims are sometimes used as a method for transferring the load from stringers to cap beams. Center loading strips aid in transferring the vertical reaction load from stringer to cap concentrically. This prevents the stringer bottom flange from bearing on the flange edges of the cap. Otherwise the stringer can induce torsional rotation in the cap if the stringer bears on points other than the center of the cap. It is critical that center loading strips are symmetrically located about a vertical line that passes through the webs of both the stringer and the cap. This ensures the transfer of the force reactions from stringer to the cap through the web of cap thereby preventing any unintended moment on the cap. See Figure 4-4. *Center Loading Strip Details*.

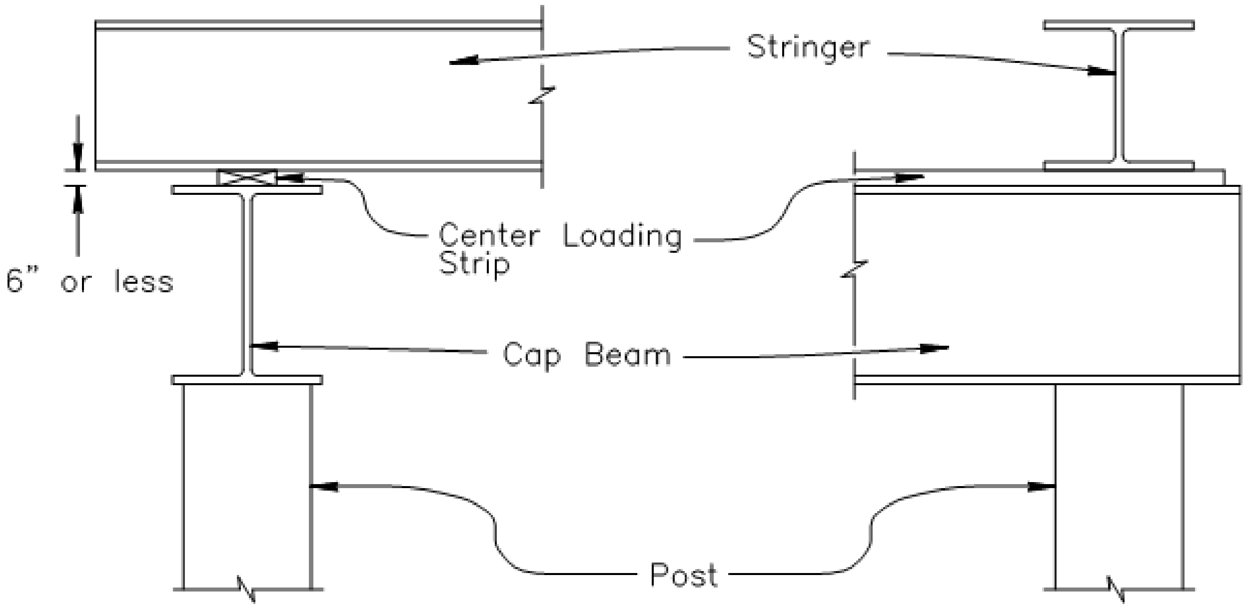


Figure 4-4. Center Loading Strip Details

4-4.02 Design Criteria

The maximum thickness of loading strips or shims must not exceed:

- 6 inches
- This limit also applies to multiple built-up strips or shims. This maximum thickness limitation eliminates excessive build-up between the cap and the stringer beam that could lead to stability problems

The allowable compressive stress for perpendicular-to-grain loading for center loading strips is:

- 900 psi for single strip or shim up to 6 inches thick.
- Comply with National Design Specifications (NDS) for multiple strips or shims with a combined thickness up to 6 inches thick.

4-5 Construction Sequence

4-5.01 Introduction

Unless a concrete placing sequence is shown on the bridge plans, the shop drawings must include a placing diagram showing the proposed placing sequence and the location of all construction joints. If a placing schedule or sequence is shown on the plans, no deviation is permitted, and the falsework must be designed and constructed to accommodate the planned placing sequence.

4-5.02 Transverse Construction Joint

The location of transverse construction joints in the bridge superstructure is an important falsework design consideration. If a construction joint is located near the mid-point of a falsework beam, the initial concrete pour on one side of the joint will deflect the beam as the concrete dead load is applied. Later, as concrete is placed on the opposite side of the joint, additional beam deflection will occur. The additional beam deflection leaves the first concrete placed to be unsupported, and this can result in unanticipated detrimental stresses and even cracking in the permanent structure. To avoid this condition, transverse construction joints in the bridge superstructure should be designed and constructed in such a manner that subsequent pours will not produce additional stresses in the concrete already in place. In many cases the exact location of a construction joint is not critical, and the joint can be moved a few feet in either direction to accommodate the falsework design. The important point, however, is that the joint location be considered in the falsework design with respect to falsework beam span, thus avoiding a problem during construction.

4-6 T-Beam Bridges

When relatively long falsework spans are used to support T-beam structures, the added weight of the deck concrete, which often exceeds the weight of the stem, loads the stem and the falsework as the deck concrete is placed. This can produce stresses of considerable magnitude in the concrete and reinforcing steel in the girder stem.

To prevent overstressing of concrete and reinforcing steel in the girder stems of T-beam girder bridges, the *Standard Specifications*, Section 48-2.02B(1) *General* limit the length of falsework spans to:

$$\mathbf{L = 8.5T + 14} \qquad (4\text{-}6\text{-}1)$$

where **L** = Length of falsework span (ft)

T = Depth of T-beam girder measured from top of deck to bottom of girder (ft)

Occasionally contractors request to use a longer falsework span than allowed by the specifications. It is acceptable to exceed the specified span length provided the criteria in Section 5-8, *Longer T-Beam Falsework Spans,* are satisfied.

4-7 Friction

4-7.01 Introduction

Friction used as a means of resisting opposing horizontal forces is a very intangible factor. The use of friction for this purpose should be considered with caution. The coefficient of friction should be assumed as not being greater than 0.30 maximum for all contact surfaces. The only exception to this friction factor is for concrete anchor blocks, see Section 5-5.13A, *Cable Anchored to Concrete Blocks*.

In general, friction may be considered as resisting the tendency of one member to slide over or across another member, provided frictional resistance is actually developed under the loading condition being investigated.

Do not consider frictional resistance in any case where the dead load is not applied uniformly through all stages of construction, or where continuity would reduce the load acting on a support under a non-uniform loading condition.

Do not consider frictional resistance as contributing to the lateral stability of beams or stringers. If flange support is required, the method of support must be positive and independent of any theoretical frictional resistance.

Do not consider friction as contributing to the resisting capacity of any connecting device unless the device is specifically designed and marketed as a friction-type connector, except as otherwise provided in the Section 7-3.06, *C-Clamps*.

4-8 Prestressing Forces

When cast-in-place prestressed structures are stressed, the initial stressing produces an upward deflection in the positive moment area, and the resulting redistribution of vertical forces transfers the superstructure dead load from the falsework to the adjacent abutments and columns.

The St*andard Specifications,* Section 48-2.02B(2), *Design Criteria – Loads,* require that the falsework must support any increase or readjustment of loads caused by prestressing forces.

An example of dead load redistribution due to longitudinal prestressing is stage construction of continuous bridges with hinges. For these bridges, prestressing will reduce the dead load on the falsework near the center of the suspended span and increase the load on the falsework at the hinge. The forces involved in the dead load redistribution are of considerable magnitude, since up to 3/8 of the total suspended span dead load may be transferred to the falsework at the hinge. The load due to dead load transfer must be added to the dead load calculated in the usual manner to obtain the total dead load for the falsework design at the hinge support.

If the dead load hinge reaction (the load applied to the cantilever span by the supported span) is not shown on the contract plans, it may be obtained from the designer.

The effect of transverse prestressing is a falsework design consideration. If the structure is designed to include transverse prestressing of decks or caps, the project plans will include the stressing sequence, and the falsework must be designed to accommodate the sequence outlined on the plans.

4-9 Long-Term Superstructure Deflection

Depending on factors, such as the length of time the falsework is to remain in place and the method and sequence of removal, long term deflection of the bridge superstructure occurring after prestressing may be a design consideration.

Long term superstructure deflection will begin as soon as the structure is stressed. As deflection occurs, a portion of the dead load initially transferred to the falsework at the hinge will be carried back to the falsework near the center of the span. The amount of dead load carried back is a function of time and is not easy to predict. However, this should not present a problem in most cases because the load carried back cannot exceed the load originally resisted by the falsework.

If falsework is removed in stages, field engineers should be aware that part of the redistributed load will be carried back with time, and that components of the falsework system remaining in place near the center of the span will be subjected to a gradually increasing load as superstructure deflection takes place. Dead load carried back may be an important consideration when evaluating the adequacy of a given falsework removal sequence. See also Section 9-5.03, *Stage Construction,* for removal sequence considerations.

4-10 Falsework at Deck Overhangs

4-10.01 Introduction

For box girder structures with cantilevered deck overhangs, the normal 2-stage construction sequence results in differential loading of the exterior and first interior falsework beams. The differential loading condition is exacerbated if the exterior girder is also sloping outward at the top, as is usually the case. Depending on the beam size and location, differential loading may result in differential beam deflection, causing the exterior girder stem to rotate. Girder rotation may occur during the girder stem pour or during the deck pour, or during both pours.

4-10.02 Stem and Soffit Pour

Referring to Figure 4-5(a), *Deck Pour Differential Deflection – Prior to Deck Pour*, during a girder stem pour, stringer A may deflect more than stringer B, causing a downward movement of stringer A relative to point C. This downward movement of stringer A causes the girder-stem form to rotate inward. Inward rotation will affect alignment and grade at the top of the girder stem and will cause the rebar in the stem and soffit corner to move during the pour. If the inward rotation is small, minor adjustments can be made to the grades before the deck pour and the movement of the rebar will be negligible. If the inward rotation is large, it may not be possible to make the grade adjustments before the deck pour and the rebar in the stem and soffit corner may move out of place and create separation during the pour. The effects of differential stringer deflection during the girder stem pour must be investigated as a precautionary measure to determine whether any adverse consequences will occur. If the soffit is placed separate from the stem, it may be necessary to realign the stem forms before placing the stem concrete.

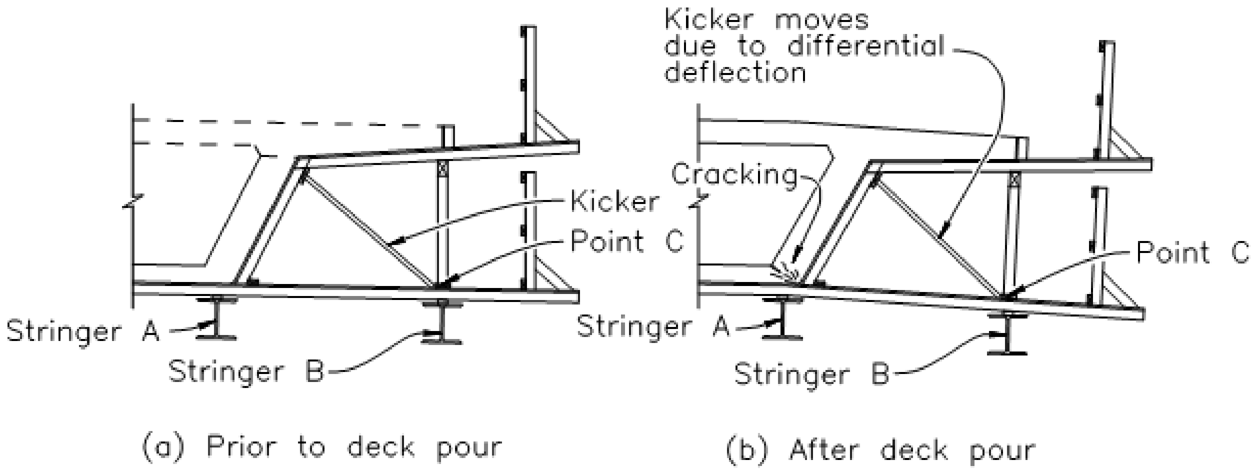


Figure 4-5. Deck Pour Differential Deflection

4-10.03 Deck Pour

Referring to Figure 4-5, *Deck Pour Differential Deflection,* a situation may develop during the deck pour where the weight of the deck overhang may cause stringer B to deflect more than stringer A. This differential deflection causes a downward movement at point C relative to stringer A, which pulls the kicker away from the girder stem form panel and leaves the sloping exterior girder unsupported causing it to rotate outward. This will induce torsional stresses in the concrete and reinforcing steel at the girder base. This outward rotational moment is exacerbated by the weight of the deck concrete on the inside of the exterior girder.

The load applied to the exterior and adjacent interior falsework beams, during the deck pour, should be investigated in all cases where the depth of a box girder structure, having sloped exterior girders, exceeds 5 feet. When the applied loads result in differential beam deflection of sufficient magnitude to cause the exterior girder support system to become dysfunctional, the falsework design must include means to resist girder rotation. The method by which this is accomplished must be shown on the shop drawings, such as tiebacks to the base of the adjacent interior girder.

4-11 Concrete Deck on Steel Girders

4-11.01 New Steel Girder Bridges

The *Standard Specifications*, Section 55-1.03B, *Steel Structures – Falsework,* include special requirements for falsework supporting the concrete deck on steel girder bridges. These requirements are included to control the manner falsework loads are applied to the steel girder, and thus prevent undesirable distortion of the permanent structure. See Figure 4-6, *Steel Girder Falsework*.

Horizontal loads applied to the girder flanges by the falsework will produce a torsional moment in the girder. To prevent possible overstressing of the permanent diaphragm connections, the falsework design must include temporary struts and/or ties to resist the full torsional moment and to prevent appreciable relative vertical movement between the edge of deck form and the adjacent steel girder.

The falsework must be designed and constructed to comply with *Standard Specifications,* Section 55-1.03B, *Steel Structures – Falsework, as follows*:

- Any loads applied to the girder web must be applied within 6-inches of a flange or stiffener.
- Temporary struts must be provided as necessary to resist lateral loads.
- The applied loads must be distributed to prevent local distortion of the web.

Standard Specifications, Section 55-1.02E(7)(a), *Steel Structures – Fabrication – Welding – General,* do not allow welding or tack welding of brackets, clips, shipping devices, or any other material not described to any part of the girders unless it is shown on the girder shop drawings.

Figure 4-6, *Steel Girder Falsework*, shows conventional falsework for the interior deck forms, however, it is common to use stay in place steel deck forms for the interior deck.

The top flange of the steel girders must be surveyed after the girders are erected, but before any load is placed on them. This step is necessary to verify that the theoretical girder dead load camber shown on the bridge contract plans was achieved. Any deviation in the camber must be accounted for by adjusting the thickness of the fillet between the top of the girder flange and the bottom of the deck.

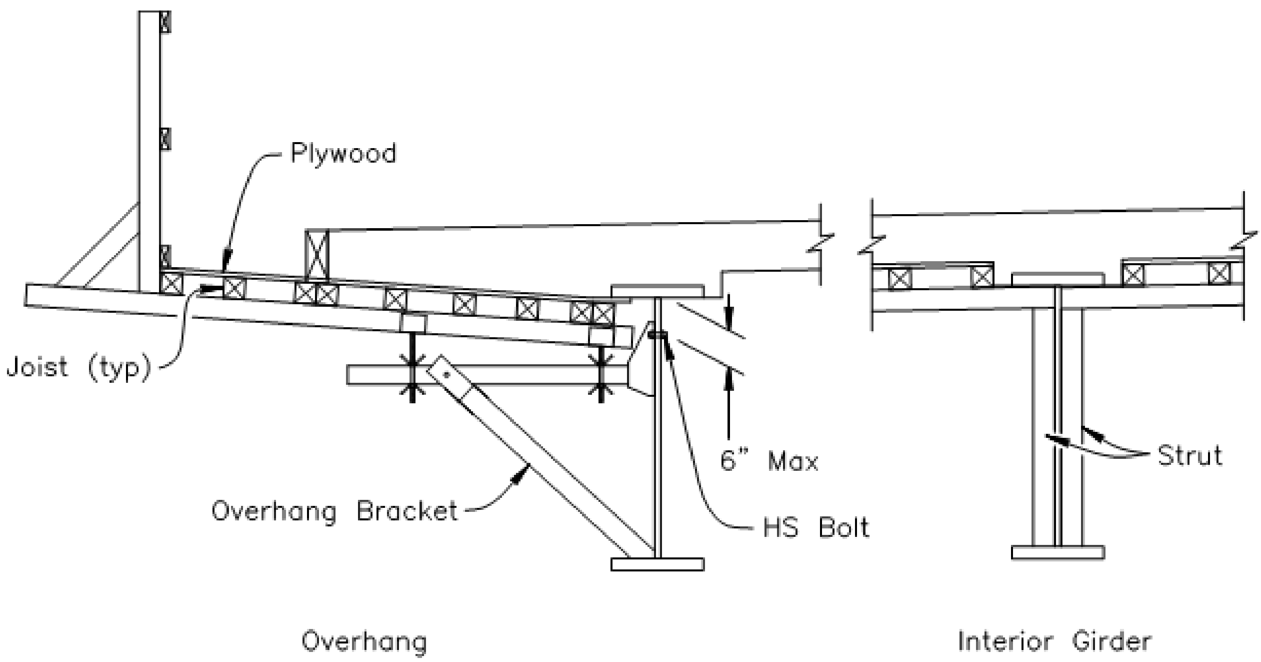


Figure 4-6. Steel Girder Falsework

4-11.02 Steel Girder Widenings

In addition to the requirements stated in the previous section, special attention must be given to deck falsework on widenings. A steel girder bridge widening must be constructed as an independent "bridge" and then tied to the existing bridge with the closure pour.

Referring to Figure 4-7, *Steel Girder Widening Falsework*, the falsework on the new girder adjacent to the existing must be constructed independently of the existing bridge. Typically, it is constructed using overhang brackets to support the portion of the deck hanging over the new girder next to the closure pour.

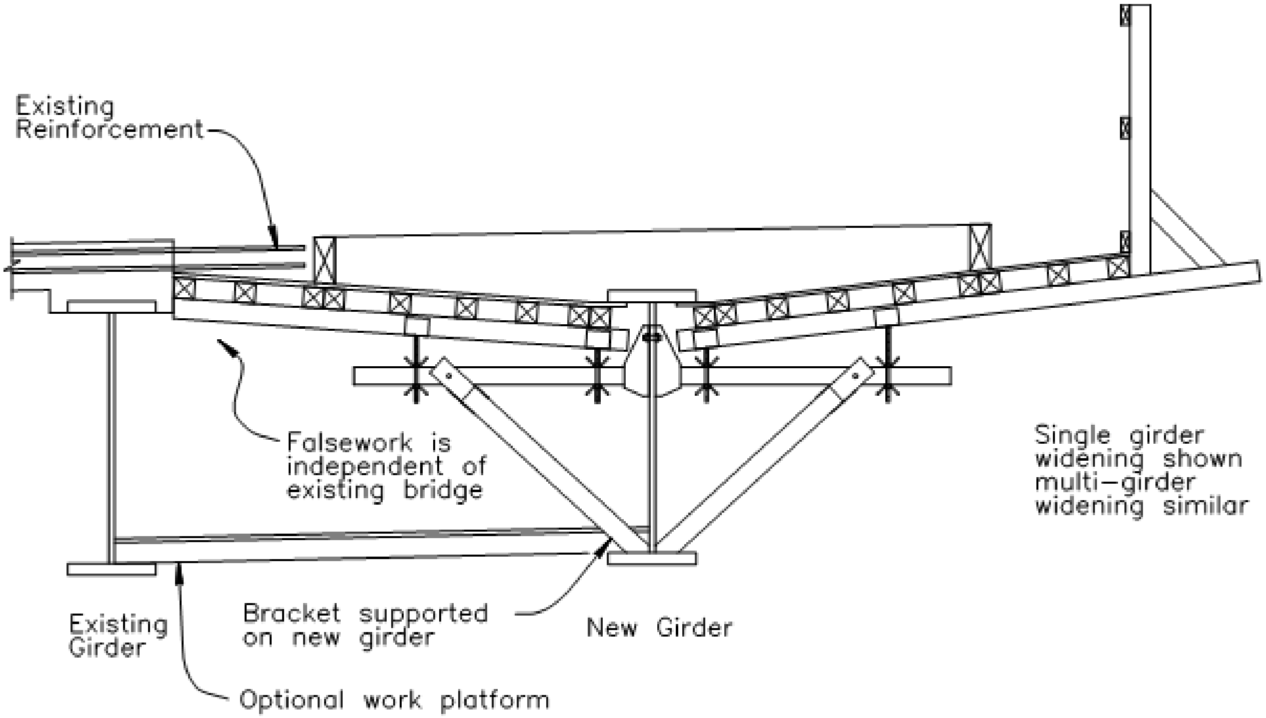


Figure 4-7. Steel Girder Widening Falsework.

Referring to Figure 4-8, *Incorrect Steel Girder Widening Falsework*, the portion of the new deck, hanging over the new girder towards the closure pour, must not be supported by the existing girder. The existing girder is in its final position, but the new girder will deflect downward during the deck pour. The deflection can be up to several inches depending upon the span length and girder stiffness.

If this portion of the deck is supported on the existing girder, the deck form between the new and existing girder will continue to move as the deck pour proceeds along the new girder and the new girder continues to deflect. This will also result in the following:

- Point A: The slope on the new deck will change
- Point B: Rebar in the concrete placed earlier in the pour will keep moving resulting in debonding, e.g. when the pour is at mid-span, the forms and rebar at 1/4-span are still moving due to the girder deflection.
- Point C: The concrete may debond from the steel girder.
- Point D: The outer screw jack will deflect down the same amount as the inner screw jack, but the joist will move less at the outer screw jack, causing the joist to only be supported at the inner screw jack and the post on the existing girder and hence the joist will be over loaded.

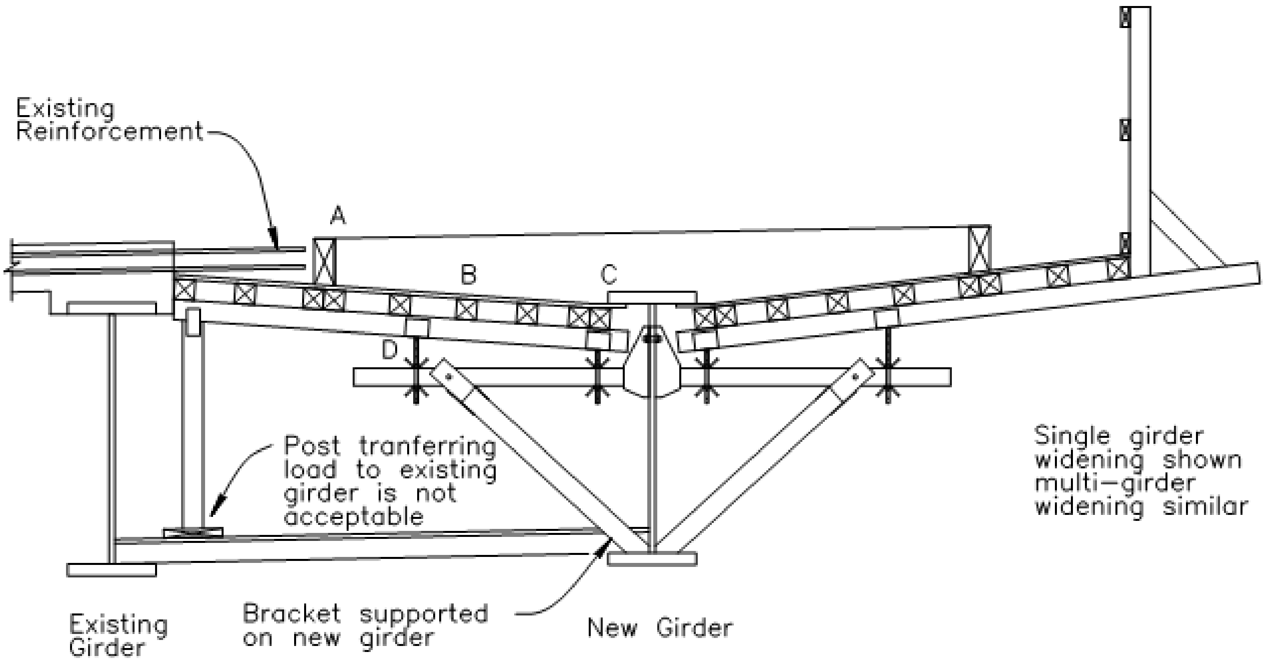


Figure 4-8. Incorrect Steel Girder Widening Falsework.

The top of the steel girders must be surveyed after the girders are erected, but before any load is placed on them. This step is necessary to verify that the theoretical girder dead load camber shown on the bridge contract plans was achieved. Any deviation in the camber must be accounted for by adjusting the thickness of the fillet between the top of the girder flange and the bottom of the deck.

4-12 Falsework Over or Adjacent to Roadways or Railroads

4-12.01 Introduction

The *Standard Specifications*, Section 48-2.02B(4), *Falsework – Design Criteria – Special Locations,* include special requirements, which apply only to falsework over or adjacent to roadways and railroads that are open to traffic. These requirements are included to ensure higher standards of design and construction at locations where public safety is involved.

Falsework posts are adjacent to roadways or railroads if:

- The post supports members that cross over roadways or railroads.
- The post is located such that the horizontal distance from the traffic side of the falsework to the edge of pavement or to a point 10 feet from the centerline of

railroad track is less than the height of the falsework and forms. See Figure 4-9, *Falsework Adjacent to Roadways or Railroads*.

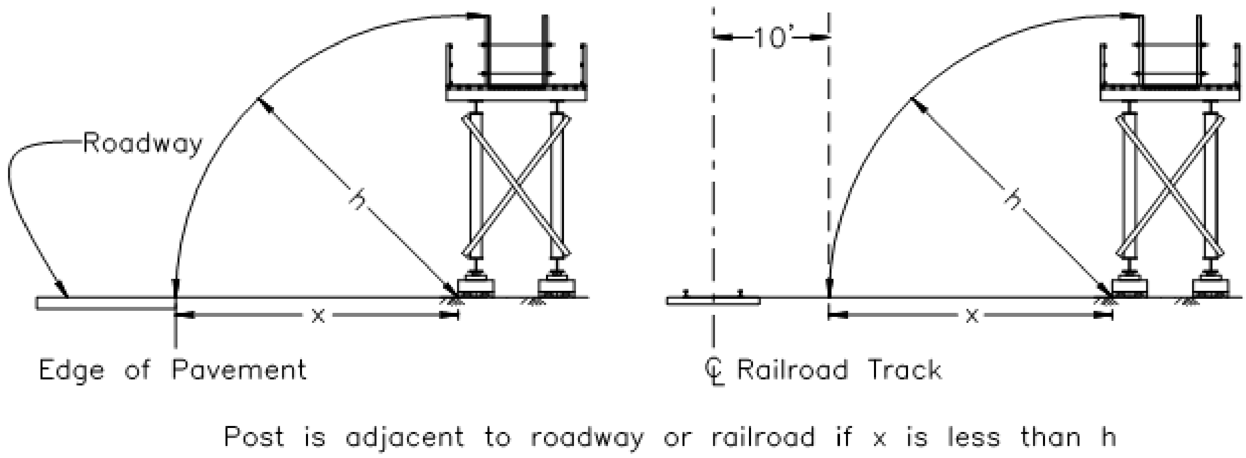


Figure 4-9. Falsework Adjacent to Roadways or Railroads.

4-12.02 Falsework Openings

Whenever an operation will reduce clearances available to public traffic, the *Standard Specifications,* Section 7-1.04, *Public Safety,* require the contractor to notify the resident engineer within a specified timeframe before the anticipated start of the operation. Moreover, the *Standard Specifications*, Section 12-4.02A(3)(b), *Traffic Control Systems – Submittals – Closure Schedules*, require 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 Bridge Construction Memo 7-1.04, *Impaired Clearances at Falsework Traffic Openings,* the structure representative completes form SC-4103, *Report of Falsework Clearance,* from the shop drawings. With this information, the structure representative then completes form TR-0019, *Notice of Change in Clearance or Bridge Weight Rating,* or TR-0029, *Notice of Change in Clearance or Bridge Weight Rating,* as applicable. After the Bridge Construction Engineer has reviewed and initialed the appropriate form, 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 falsework, the structure representative verifies the clearance.

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

The width of a vehicular opening is the distance between the temporary railings. The clear distance between falsework posts will be considerably greater than the width shown in the special provisions.

For a vehicular opening, no portion of the falsework may encroach into the clearance zone shown in Figure 4-10, *Clearance to Railing Members and Barriers*.

4-12.03 Horizontal Clearance

Horizontal clearance for falsework openings and any falsework adjacent to roadways must be at least those shown in Table 4-1, *Clearance to Railing Members and Barriers,* and Figure 4-10, *Clearance to Railing Members and Barriers*:

Table 4-1. Clearance to Railing Members and Barriers.

Falsework Member	Clearance to railing members, barriers, and anchored temporary railing	Clearance to unanchored temporary railing
Footings (incl. pads and corbels)	0'-3"	2'-0"
Piles (incl. pile bents)	1'-0"	2'-9"
Other members	2'-0"	2'-9"

The clearance is measured from the portion of the railing or barrier closest to the falsework, e.g. for temporary railing (Type K) it is measured from the toe.

Corbels are considered part of the footing or foundation. However, if corbels are used as build up, they are considered as other members.

For K-rail anchor details see *Standard Plans,* T3B, *Temporary Railing (Type K)*.

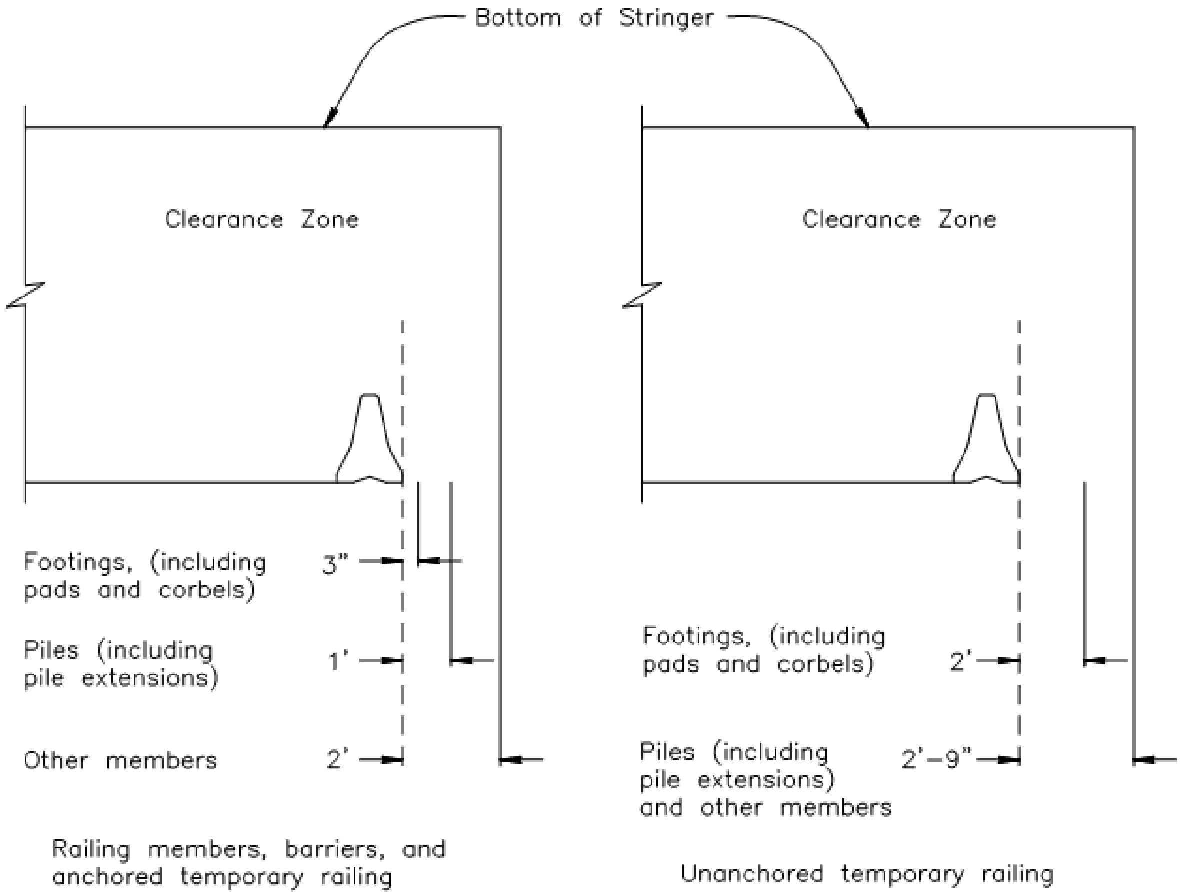


Figure 4-10. Clearance to Railing Members and Barriers.

4-12.04 Vertical Clearance

Calculating anticipated vertical clearance prior to falsework erection and measuring actual vertical clearance after erection is required to ensure Transportation Permits Branch is aware of impaired clearances. Transportation Permits Branch uses this information to issue transportation permits.

Example 1: *Clearances at Falsework Openings,* in Appendix D, *Example Problems,* illustrates proper methodology for calculating the anticipated minimum vertical clearance. The following specific factors should be considered:

- Stringer bottom flange elevations over the roadway during all stages of construction. Measure the clearance to the lowest stringer over the roadway.
- Verify if the point of minimum vertical clearance is over the roadway or behind the K-Rail during construction.
- Whether or not falsework is skewed, etc.

Deflection of the falsework stringers under the dead load of the concrete will reduce the theoretical clearance, and this must be considered in the design.

Figure 4-11, *Points of Minimum Vertical Clearance,* shows that the point of minimum *final* vertical clearance of the bridge may not be the same as the point of minimum *temporary* vertical clearance for the falsework during construction.

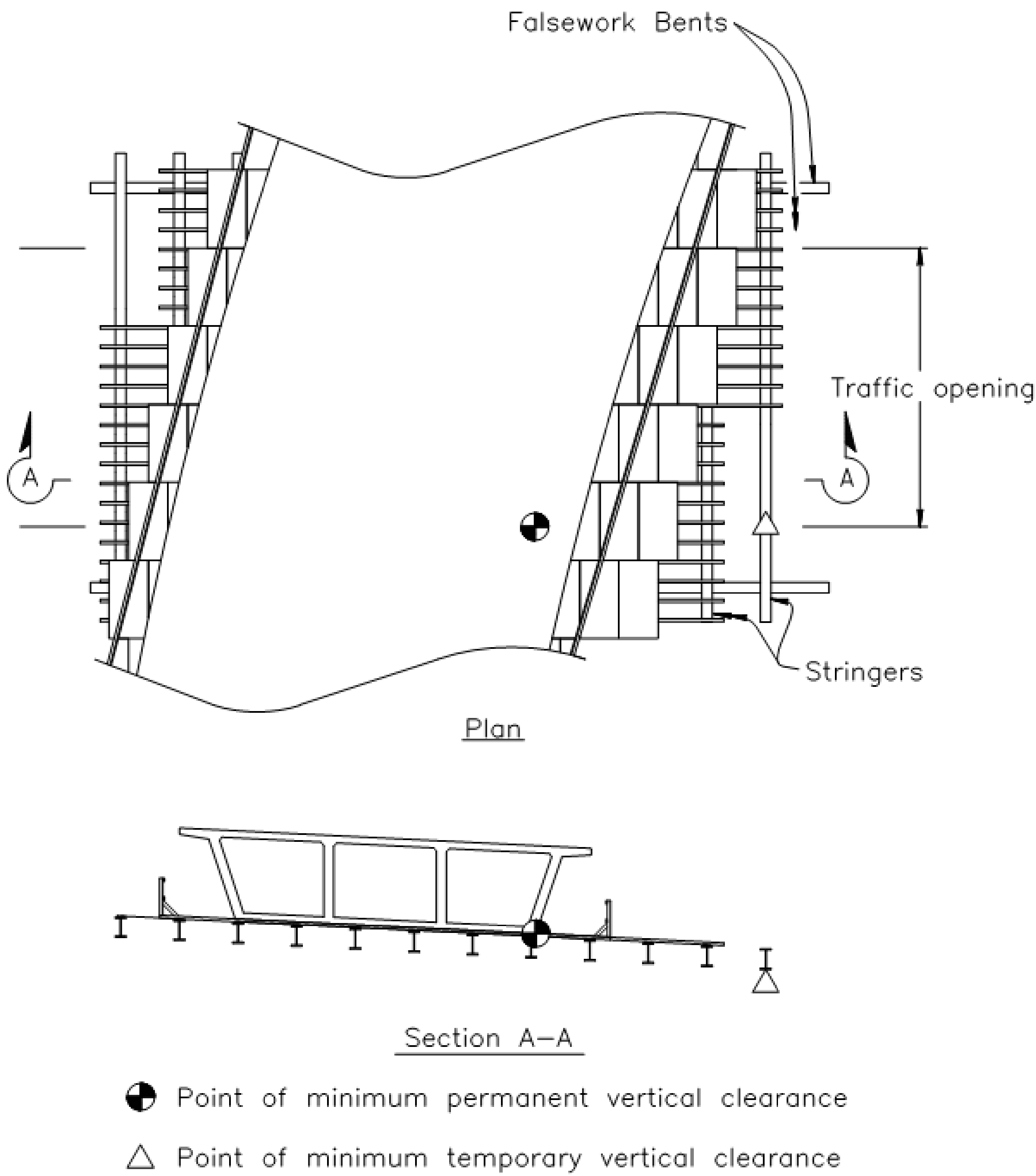


Figure 4-11. Points of Minimum Vertical Clearance.

4-12.05 Requirements Over or Adjacent to Roadways or Railroads

4-12.05A Introduction

Refer to *Standard Specifications*, Section 48-2.02B(4), *Falsework – Design Criteria – Special Locations*. The special requirements discussed in this section apply to falsework over or adjacent to roadways and railroads and within the limits shown in Section 4-12.01, *Introduction*. Additional requirements that apply only to falsework over or adjacent to railroads are discussed in Section 4-12.06, *Additional Requirements Over or Adjacent to Railroads*.

Similar requirements also apply to falsework with bents perpendicular to or at an angle to roadways and railroads, which are not supporting members over the roadway or railroad, as explained in the following sections.

4-12.05B Post Material and Parameters

Falsework posts must be either:

- Steel with a minimum section modulus of 9.5 in^3 about each axis
- Timber with a minimum section modulus of 250 in^3 about each axis

When pipe frame or tubular steel components are used in falsework over or adjacent to a roadway or railroad, either as individual posts or as legs in a tower bent, the specified minimum section modulus for steel posts will apply to the post or tower leg, but not to the screw jack extension.

4-12.05C Impact Loads and Mechanical Connections

Each falsework post must be mechanically connected to its supporting footing or otherwise laterally restrained and comply with the following:

- 2000 lb force applied at the base of the post is:
  - Applied at the base of each post regardless of its size, spacing, or loading; however, it will be assumed as acting on only one post at a time.
  - Applied in any direction except toward the roadway or railroad.
- Lateral restraint must be effective as shown in Figure 4-12, *Application of 2000 Pound Load*.
- For a bent in a highway median or in between railroad tracks, the restraint must be effective in all directions.

- For falsework with bents parallel or at an angle to roadways or railroads, this requirement applies to all posts within the limits shown in Section 4-12.01, *Introduction*, but not less than two posts for angled bents.

In the *Standard Specifications*, Section 48-2.02B(4), *Falsework – Design Criteria – Special Locations*, the term support footing means the element of the falsework system that is set on the ground.

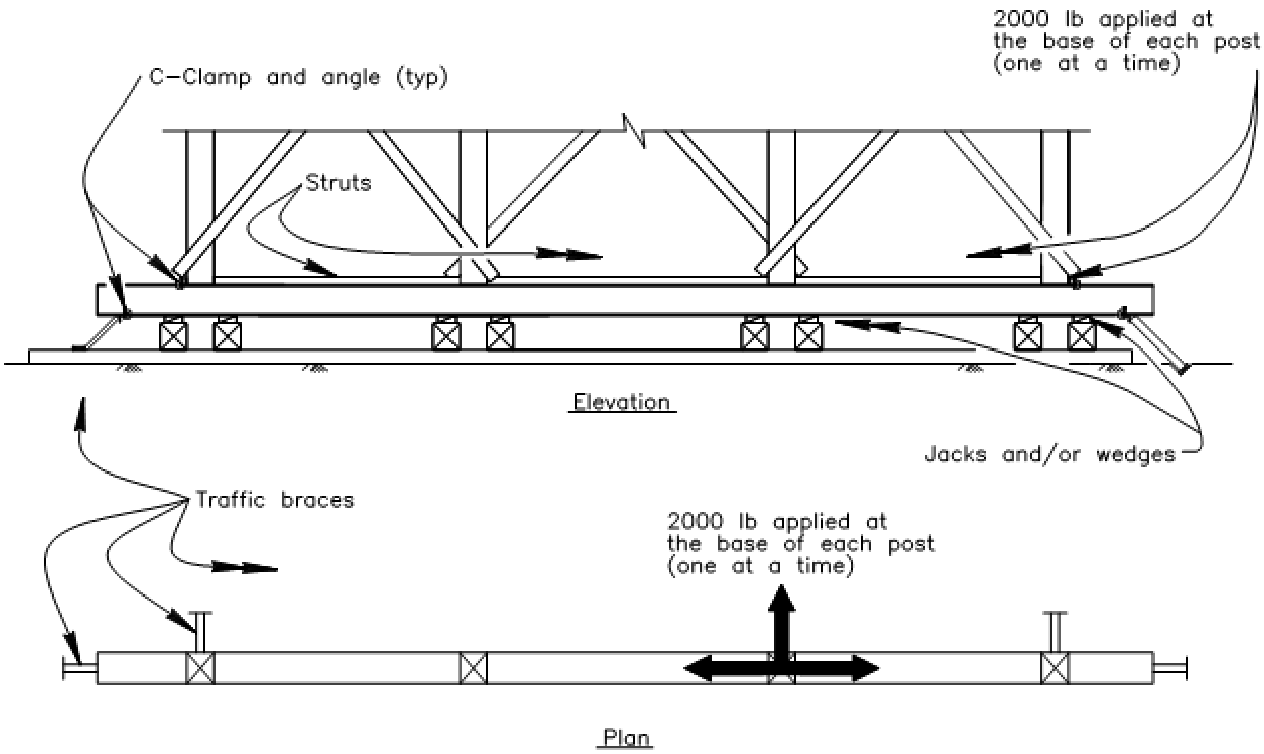


Figure 4-12. Application of 2000 Pound Load.

Many contractors prefer to adjust falsework to grade by wedging or jacking at the bottom of a falsework bent, rather than at the top. In such designs, two or more posts will be supported by a bottom cap, which will be supported by wedges or wedges over sand jacks set on the falsework footings.

Each post must be mechanically connected to the bottom cap to withstand a force of at least 2000 lb. The design force does not accumulate along the bottom cap, so the connection between the bottom cap and the falsework foundation is only required to resist 2000 lbs total, regardless of the number of posts supported. A single point of restraint will not provide adequate resistance in the transverse direction when the 2000 lb force is applied perpendicular to the beam, unless the connection is capable of resisting moment as well as shear. The most practical solution is to restrain the bottom cap at both ends, and both connections must be designed to resist (transfer) 2000 lbs.

As an alternative means of providing lateral restraint, the 2000 lb force may be carried from the bottom cap directly to the ground in the manner shown in Figure 4-12, *Application of 2000 Pound Load*.

If cap systems are used as discussed in Section 6-8, *Cap Systems*, all the caps in the cap system must be restrained for the 2000 lb load as discussed above for the single bottom cap.

Each falsework post must be mechanically connected to the top cap, and the connection must be designed to resist and comply with the following:

- 1000 lb force applied at the top of the post will be:
  - Applied at the top of each post regardless of its size, spacing, or loading; however, it will be assumed as acting on only one post at a time.
  - Applied in any direction.
- Lateral restraint must be effective in any horizontal direction.
- For a bent in a highway median or in between railroad tracks, the restraint must be effective in all directions.
- For falsework with bents parallel or at an angle to roadways or railroads, this requirement applies to all posts within the limits shown in Section 4-12.01, *Introduction*, but not less than two posts.

Standard Specifications, Section 48-2.02B(4), *Falsework – Design Criteria – Special Locations,* requires certain stringers to be mechanically connected to the cap. The connection must be designed to resist and comply with the following:

- 500 lb. force applied in any direction, including uplift
- These connections must be installed and functional before traffic is permitted to pass under the falsework span.
- For falsework stringers parallel or at an angle to roadways or railroads, where traffic does not pass under the stringers, but the stringer is within the limits shown in Section 4-12.01, *Introduction*, mechanically connect the exterior stringer to the cap. The connection must be capable of resisting a force in any direction, including uplift, of not less than 500 lbs.

Details showing the connection between stringer and cap, cap and post, and post and footing, must be shown on the shop drawings. All components must be designed so that the specified maximum allowable stresses in bending, shear, and bearing are not exceeded. The load duration factor, $\mathbf{C_D}$, for impact loading may be used to determine the allowable value of nails and bolts used in the connection:

- C_D = 1.6 for nailed and bolted connections only

4-12.05D Bracing

The *Standard Specifications*, Section 48-2.02B(4), *Falsework – Design Criteria – Special Locations,* require bolted connections when timber members are used to brace falsework bents over or adjacent to roadways and railroads:

- For falsework with bents parallel or at an angle to roadways or railroads, this requirement applies to all bracing within the limits shown in Section 4-12.01, *Introduction*, but not less than one cross brace between two posts.
- This requirement applies to bracing in the longitudinal as well as the transverse direction.
- The bolt diameter must be at least 5/8-inch.
- Substitution of bolts with coil rods is permitted if the root diameter of the coil rod is greater than or equal to the required bolt diameter. Also, the substituted coil rods must provide the capacity required for the connection. The term coil rod includes threaded coil rods, as well as threaded rods.
- The brace must be bolted at both ends. It is not acceptable practice to use a bolt at one end of a brace and nails or lag screws at the other end.
- Other fastener types may be used as intermediate fasteners in the center intersection of a diagonal brace. See Section 6-3.02, *Wood Cross Bracing.*

All components of the falsework system which contribute to horizontal stability and resistance to impact, except for bolts in bracing, must be installed at the time each element of the falsework is erected, see *Standard Specifications*, Section 48-2.03C, *Falsework – Erection*. Therefore, friction cannot be considered as contributing to the strength of the connection, at either the top or the bottom, because frictional resistance is not developed until a load is applied.

Bolts in bolted connections need not be installed when the falsework is erected to facilitate adjusting of the falsework to grade. However, if the contractor elects to use nails in lieu of bolts as a temporary expedient, the nailed connection must be shown on the shop drawings, and the connection must be designed to resist the assumed horizontal load while the connection is in use.

When nails are used as temporary fasteners to facilitate grade adjustment, they should be replaced by bolts as soon as feasible, and in any case prior to placing concrete.

4-12.05D(1) Temporary Bracing

The *Standard Specifications*, Section 48-2.03A, *Falsework – Construction – General,* require the installation of temporary bracing as necessary to withstand all imposed loads during erection, construction, and removal of the falsework. While wind loads are to be considered in the design, the basic requirement is that the bracing must be adequate to "withstand all imposed loads." Under the specifications, ~~then~~, the contractor must determine the design load, which may not be less than the specified wind load for the height of falsework under consideration.

Details showing the temporary bracing, or other means of support provided to meet the intent of the specifications, must be shown on the shop drawings. Such details are a part of the design and must comply with all contract requirements even though the bracing or other means of support may be only "temporary" restraining devices.

4-12.05E Modified Design Load

The vertical load used for the design of posts and towers which support falsework adjacent to or over roadways and railroads must be modified to increase the factor of safety. This modified design load must comply with the following:

- 150% of the load calculated in the usual manner
- Applied to post loads only
- For falsework with bents parallel or at an angle to the roadways or railroads, this requirement applies to all posts within the limits shown in Section 4-12.01, *Introduction*, but not less than two posts.

In the case of towers, the modified design load will be applied to all tower legs if any of the tower legs are within the limit shown in Section 4-12.01, *Introduction*.

If the load on falsework adjacent to or over a roadway or railroad will be increased by load transfer due to prestressing, the design vertical load for posts and towers will be either the actual (unmodified) load plus the additional load due to prestressing or 150% of the design load, whichever is greater.

4-12.06 Additional Requirements Over or Adjacent to Railroads

4-12.06A Introduction

In addition to the requirement in Section 4-12.05, *Requirements Over or Adjacent to Roadways or Railroads*, the design of falsework which is over or adjacent to railroads must comply with all the special requirements in this section as well. Moreover, the design must also comply with the current railroad guidelines.

4-12.06B Mechanical Connections

All falsework stringers that span over a railroad must be mechanically connected to the caps. The mechanical connection must be capable of resisting a 500 lb. load in any direction, including uplift on the stringer.

4-12.06C Bracing

The principal design requirement is that bracing for falsework bents located within 20 feet of the track centerline must be designed to resist the following:

- 5000 lb. or the assumed horizontal load, whichever is greater
- This requirement applies to both transverse and longitudinal bracing

In the specification context, the term "bent" means the overall length of the falsework bent regardless of the number of posts used. Typically, the 5000 lb. load will govern the design only in the case of relatively narrow structures where the bent consists of five, or fewer, falsework posts.

The load duration factor, $\mathbf{C_D}$, in the wood connections is determined as follows:

- 1.25 for the assumed horizontal load or 5000 lb. load
- 1.6 for wind load

4-12.06D Timber

For timber members:

- All connections must be bolted
- The bolt diameter must be at least 5/8-inch
- Substitution of bolts with coil rods is permitted if the root diameter of the coil rod is greater than or equal to the required bolt diameter. Also, the substituted rods must provide the capacity required for the connection

The railroad will require solid end blocking when timber stringers are used regardless of the height-to-width ratio of the timber stringers.

The load duration factor, $\mathbf{C_D}$, in the wood connections is determined as follows:

- $\mathbf{C}_D$ = 1.25 for the assumed horizontal load or 5000 lb load
- $\mathbf{C}_D$ = 1.6 for wind load

4-12.06E Steel

For steel, the allowable compression, tensile, bending, and shear stresses are limited to:

$$\mathbf{F_b} \leq \mathbf{0.55F_y} \quad \text{(4-12.06E-1)}$$

$$\mathbf{F_v} \leq \mathbf{0.35F_y} \quad \text{(4-12.06E-2)}$$

where $\mathbf{F_b}$ = Maximum allowable compression, tensile, and bending stress

$\mathbf{F_v}$ = Maximum allowable shear stress

$\mathbf{F_y}$ = Minimum yield strength

4-12.06F Shop Drawings Over or Adjacent to Railroads

The design of falsework over or adjacent to railroads is subject to review and approval by the railroad company involved. To expedite approval, shop drawings submitted for railroad company review should conform to the following procedural requirements:

- All design and construction details must be shown. If a reference is made to a standard plan or to a detail shown on a previously submitted drawing for another structure in the contract, such plans or drawings must accompany the submittal to the railroad.
- When submitting only that portion of the falsework which is over or adjacent to the railroad, details of the adjacent falsework spans must be shown, as these spans will affect the design of the bents over or adjacent to the railroad.
- Design features or details for more than one structure must not be shown on the same drawing.
- The shop drawings must include a sketch showing the location of the temporary minimum horizontal and vertical clearance to the falsework.
- All falsework clearances and clearances to equipment must be clearly shown. The vertical clearance is measured from the top of the track rail and horizontal clearances are measured from the centerline of the tracks.

- All temporary structures and equipment within 25 feet of centerline of track must be shown on the shop drawings. Similarly, all temporary structures and equipment, which if they were to fall over would land within 25 feet of centerline of track, must be shown on the shop drawings.
- All temporary structures and equipment within railroad right-of-way must be shown on the shop drawings. Similarly, all temporary structures and equipment, which if they were to fall over would land within railroad right-of-way, must be shown on the shop drawings.
- Soffit and deck overhang forming details should be included

See also Section 2-4.02B. *Review Procedure when Railroad is Involved,* for additional information.

4-13 Waste Slabs

4-13.01 Introduction

A waste slab is a concrete slab finished to a smooth surface which is set to the soffit grade of a bridge superstructure. It is cast on compacted material in a fill or in a cut and becomes the bottom soffit form for the structure. On completion of the structure the fill or cut material along with the slab is removed to final cross section, see Figure 4-13, *Waste Slab*.

4-13.02 Considerations

Waste slabs are considered a construction method and not falsework as defined by the *Falsework Manual*. In order to check the adequacy of the slabs, the structure representative should require shop drawings as outlined in the *Standard Specifications*, Section 5-I.23B(2), *Shop Drawings*.

Some factors to be considered are:

- Type of material under the slab
- Amount and depth of compaction
- Load on slab
- Slab thickness
- Slab must be wide enough to support exterior girder and overhang falsework
- Finish must comply with *Standard Specifications*, Section 51-1.03C(2), *Forms* and *Standard Specifications*, Section 51-1.03F, *Finishing Concrete.*
- Bond breaker between soffit slab and waste slab

- Settlement (subsidence with time if on fill)
- Bridge camber
- Down drag on the bridge foundation piles due to fill
- Columns and abutment must be protected from stains from the fill
- Place barriers after the waste slab has been removed
- Place approach slabs after the waste slab has been removed
- Check Cal-OSHA to see if mining and tunneling regulations are applicable during fill removal after bridge is constructed

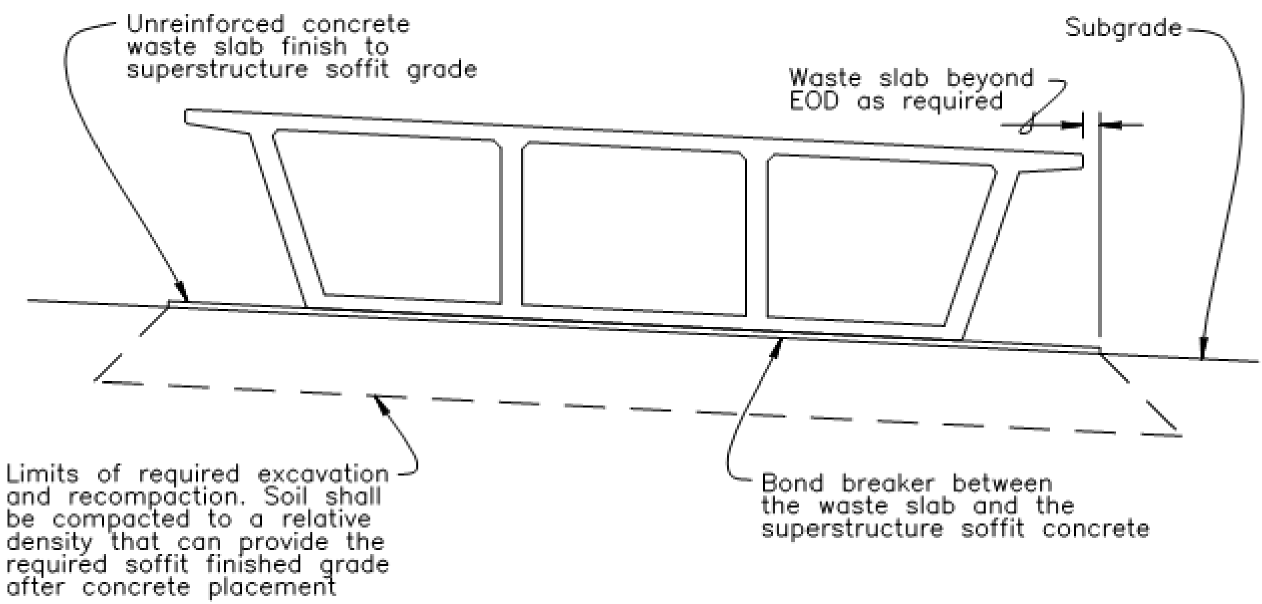


Figure 4-13. Waste Slab.

4-14 Sand Beds

4-14.01 Introduction

A sand bed is a well compacted layer of sand designed to support the soffit joists in lieu of conventional falsework. The sand bed with the soffit plywood and joist is set to the soffit grade of a bridge superstructure. The sand bed is constructed on compacted material in a fill or in a cut and becomes the support for the bottom soffit form for the structure. On completion of the structure, the fill or cut material along with the sand bed and forms is removed to final cross section, see Figure 4-14, *Sand Bed*.

4-14.02 Considerations

Sand beds are considered a construction method and not falsework as defined by the *Falsework Manual*. In order to check the adequacy of the sand beds, the structure representative should require shop drawings as outlined in the *Standard Specifications*, Section 5-l.23B(2), *Shop Drawings*.

The sand bed is constructed such that the soffit joists have full bearing on the sand bed. The plywood is designed to be supported by the joist only. Although sand is placed and compacted between the joists, this sand is only intended to help distribute the plywood load evenly but is not intended to support the plywood.

Some factors to be considered are:

- Type of material for the sand bed and under the sand bed
- Amount and depth of compaction
- Load on the sand bed from the joists
- Thickness of sand bed
- Sand bed must be wide enough to support exterior girder and overhang falsework
- Finish must comply with *Standard Specifications*, Section 51-1.03C(2), *Preparation – Forms,* and *Standard Specifications*, Section 51-1.03F, *Finishing Concrete.*
- Settlement (subsidence with time if on fill)
- Bridge camber
- Check plywood deflection based upon span between joists
- Down drag on the bridge foundation piles due to fill
- Columns and abutment must be protected from stains from the fill
- Place barriers after the sand bed has been removed
- Place approach slabs after the sand bed has been removed
- Check Cal-OSHA to see if mining and tunneling regulations are applicable during fill removal after bridge is constructed

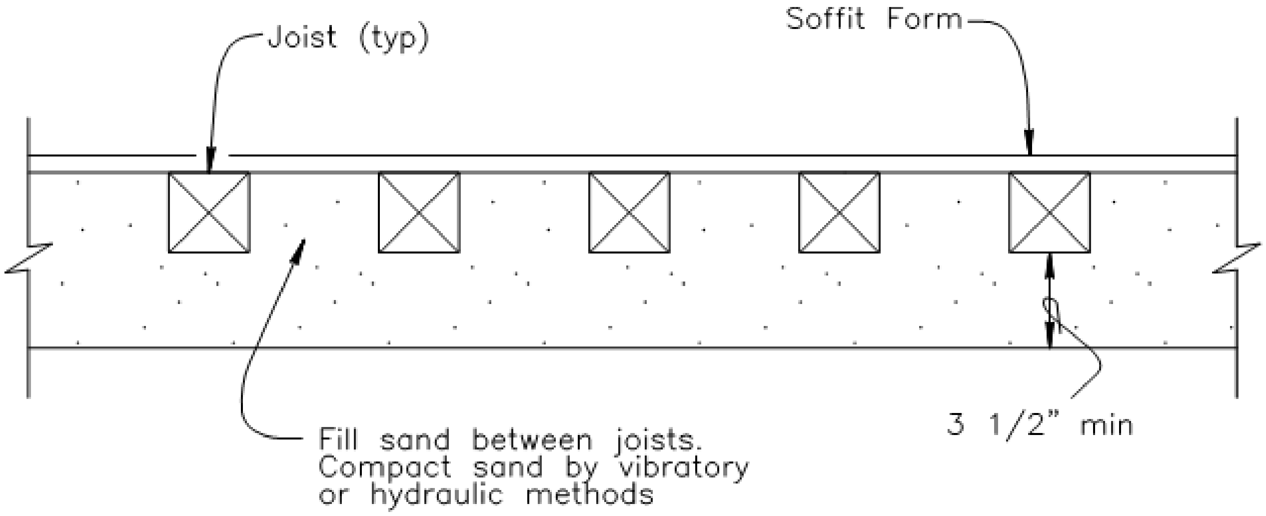


Figure 4-14. Sand Bed.

Chapter 5: Analysis

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5-1 Introduction

5-1.01 Design Requirements

The design stresses and deflections set forth in the *Standard Specifications*, Section 48-2.02B(3), *Stresses, Loadings, and Deflections,* are the maximum stresses and deflections allowed for a given loading condition. Loads are to be applied in accordance with the practice and procedures discussed in Chapter 3, *Loads*. Individual members of the falsework system, as well as the system as a whole, must be capable of resisting the specified design loads without exceeding the allowable values.

Shop drawings are not to be authorized in any case where the calculated stress or deflection in any member exceeds the allowable stress or deflection.

5-1.02 General Design Assumptions

In general, stresses in load carrying members of the falsework system may be determined by using the general formulas of civil engineering design applicable to statically determinate structures.

For some of the elements of the falsework system that are statically indeterminate, Structure Construction (SC) has developed specific methods and procedures to be used when investigating system adequacy. These procedures, which are applicable to diagonal bracing, metal shoring systems, pads, and pile bents, are explained in this manual in Chapter 6, *Stability,* Chapter 7, *Manufactured Assemblies,* and Chapter 8, *Foundations*, respectively.

The load carrying capacity of commercial products, such as jacks, beam hangers, deck overhang brackets, and other similar items, should be determined by reference to a manufacturer's published literature. The capacity can also be determined by a load test performed in accordance with Section 7-2, *Load Tests*.

The load imposed on falsework beams and stringers by the slab support system of closely spaced joists is actually applied as a series of concentrated loads. An equivalent uniform load may be assumed when calculating stresses in these members.

The effect of beam continuity must be investigated. Any theoretical advantage resulting from continuity should be neglected; however, the adverse effects must be considered to prevent overstressing of any falsework member. See Section 4-3, *Beam Continuity*.

The entire falsework system, as well as its component members, should be capable of resisting all imposed loads. The following items may contribute to the overall load to be carried by the member that is under investigation including:

- Any direct or redistributed load caused by beam continuity.
- Construction sequences.
- Prestressing.
- Deck shrinkage.
- Similar design and construction features which may contribute additional load.

5-2 Timber Members

5-2.01 Introduction

Wood differs from other building materials in that it is organic in nature, non-homogeneous, and composed of tube-like cells many times longer than they are wide. The cellular structure of wood fibers along with natural defects that develop as a tree grows are factors which result in a wide variation in the physical properties and characteristics of cut lumber. See Appendix A, *Wood Characteristics,* for a comprehensive discussion of the physical properties and characteristics of wood as a structural building material.

5-2.02 Member Size

Timber members should be assumed as S4S unless shown otherwise on the shop drawings.

The dimensions of rough cut lumber may vary appreciably from the theoretical dimension, particularly in the larger sizes commonly used in falsework construction. If the use of rough cut lumber is anticipated by the design, the actual member size must be verified prior to use.

5-2.03 Allowable Stresses and Load Duration

The allowable stresses for timber are based on the National Design Specifications (NDS), as specified in the *Standard Specifications*, Section 48-2.02B(3)(b), *Timber*. The load duration factor, $\mathbf{C_D}$, for typical falsework construction is:

- $\mathbf{C_D}$ = 1.25 based on an assumed duration of load of approximately 7 days

Occasionally a situation may occur where the falsework will be loaded for a long period of time, such as when a continuous structure is constructed in stages. In these cases, the appropriate load duration factor should be used based on the anticipated duration.

Appropriate load duration factors are discussed in Section 5-3.08, *Adjustment for Duration of Load*.

For the allowable stresses on camber strips and cap beam center loading strips, see Sections 4-2.02A, *Camber Strips,* and 4-4, *Cap Beam Center Loading Strips*, respectively.

5-2.04 Timber Beams

5-2.04A Beam Span

For simple beams, the span length is the clear distance from face-to-face of supports, plus one-half the required bearing length at each end.

For continuous beams, the span length is the center-to-center distance between supports over which the beam is continuous. For end spans of continuous beams, the span length is the distance between the center-of-bearing at the continuous support and the point of end support determined in accordance with the simple beam rule stated in the preceding paragraph.

5-2.04B Bending and Deflection

The extreme fiber stress due to bending is:

$$\mathbf{f_b} = \frac{\mathbf{Mc}}{\mathbf{I}} \quad \textbf{or} \quad \mathbf{f_b} = \frac{\mathbf{M}}{\mathbf{S}} \qquad \text{(5-2.04B-1)}$$

where $\mathbf{f_b}$ = Bending stress (psi)

M = Bending moment (in-lb)

c = Distance from the neutral axis to the extreme fiber (in)

I = Moment of inertia of the section about the neutral axis (in^4)

S = Section modulus (in^3)

Deep narrow beams may require lateral support to prevent the compression edge from buckling before the allowable bending stress is reached. See Section 5-2.04E, *Lateral Support of Wood Beams*.

The maximum deflection of a uniformly loaded simple beam is:

$$\Delta = \frac{\mathbf{5wL^4}}{\mathbf{384EI}} \qquad \text{(5-2.04B-2)}$$

where Δ = Deflection (in)

w = Uniformly distributed load (lb/in)

L = Beam span (in)

E = Modulus of elasticity (psi)

I = Moment of inertia (in^4)

5-2.04C Horizontal Shear

The general equation for horizontal shear in a rectangular beam **b** inches wide and **d** inches deep is:

$$\mathbf{f_v} = \frac{\mathbf{3V}}{\mathbf{2bd}} = \frac{\mathbf{3V}}{\mathbf{2A}} \qquad (5\text{-}2.04C\text{-}1)$$

where $\mathbf{f_v}$ = Maximum horizontal shearing-stress (psi)

V = Vertical shear (lb)

A = Cross sectional area of the beam (in^2)

b = Width of beam (in)

d = Depth of beam (in)

Theoretically, the strength of a wood beam in horizontal shear is a function of the strength property for the wood type and the extent to which a particular beam may be checked or split at the end. However, tests by the U.S. Forest Products Laboratory and others have shown that with split beams, the shear force is not uniformly distributed as assumed by the shear equation. Instead, in a split or checked beam, the upper and lower halves of the beam each resist a portion of the total horizontal shear force independently of the force resisted by the beam at the neutral axis. Therefore, a split or checked beam is capable of carrying a larger load than would appear to be the case using the general shear equation. Investigation of this phenomenon led to the derivation of so-called two-beam or checked-beam formulas from which the horizontal shearing stress may be determined with greater accuracy.

The horizontal shearing stress should be computed using equation 5-2.04C-1.[1] When computing the total shear, **V**, to use in the equation, refer to *National Design Specifications for Wood Construction* (NDS), Section 3.4.3., *Shear Design*. Neglect all

[1] The *Falsework Check* program uses the general formula for rectangular sections to calculate horizontal shear. However, this does not apply when load transfer to supporting members is with a mechanical connection.

uniformly distributed loads within a distance from the face of the support equal to the depth of the beam, **d**. Concentrated loads within a distance, d, from the supports are permitted to be multiplied x/d where x is the distance from the beam support to the concentrated load. If the allowable stress is exceeded when computed by the general equation, and if the contractor's beam design is based on the use of the checked-beam method of analysis, the shear value, **V**, may be determined by using the checked-beam formulas, and this value used in the horizontal shear calculation.

A discussion of checked-beam theory is not included in this manual because horizontal shear is seldom critical in bridge falsework spans. However, a discussion of the checked-beam method of analysis may be found in the NDS and other timber design manuals and reference is made thereto. The load within depth of the member applies to beam bearing on top of the support and not when the beam is connected with bolts/nails to the supporting member to transfer shear.

5-2.04D Compression Perpendicular to the Grain

Compression perpendicular to the grain at beam supports is:

$$\mathbf{f_{C\perp}} = \frac{\mathbf{P}}{\mathbf{A}} \qquad (5\text{-}2.04D\text{-}1)$$

where $\mathbf{f_{C\perp}}$ = Compression stress perpendicular to the grain (psi)

P = Applied load (lb)

A = Bearing area (in^2)

Initially when a beam deflects, the pressure on one edge of the support is greater than that of the other. However, wood yields enough so that the pressure equalizes and overstressing does not occur.

The use of the bearing area factor, $\mathbf{C_b}$, is permitted in the analysis of bridge falsework for small members having a bearing length, $\mathbf{L_b}$, of less than 6 inches and the contact area is 3 inches or more from the end of a supporting member, see Figure 5-1, *Effective Bearing Area*. The increase accounts for the additional wood fibers that resist the applied load after the supporting member becomes slightly indented. The bearing area factor is determined by:

$$\mathbf{C_b} = \left(\frac{\mathbf{L_b} + \frac{3}{8}}{\mathbf{L_b}}\right) \qquad (5\text{-}2.04D\text{-}2)$$

$$\mathbf{A_e} = \mathbf{AC_b} \qquad (5\text{-}2.04D\text{-}3)$$

where $\mathbf{C_b}$ = Bearing factor (in)

$\mathbf{L_b}$ = Bearing length (in)

$\mathbf{A}$ = Actual bearing area (in^2)

$\mathbf{A_e}$ = Effective bearing area (in^2)

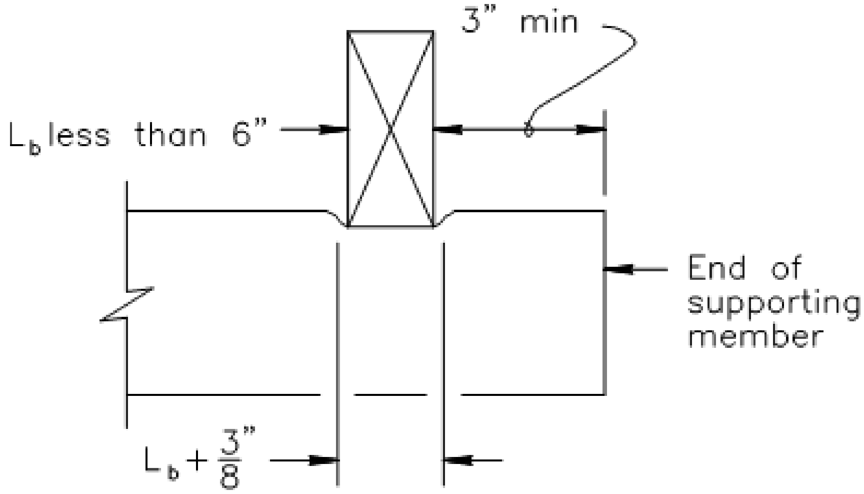


Figure 5-1. Effective Bearing Area.

To facilitate construction at locations where a conventional support may not be feasible, falsework members are occasionally supported by rods or dowels cast into a previous concrete pour. For example, lost deck forms may be supported by a ledger beam bearing on dowels cast into the girder stem. In this or any other case where a timber member bears directly on a round support, there will be some yielding of the wood fibers as the load is applied, and some crushing will occur.

Timber members supported by steel bars must comply with the following:

- Lost deck forms are supported by 2-inch nominal and wider ledger beams, bearing on either 5/8-inch or 3/4-inch diameter reinforcing bar dowels, and provided the dowel extends far enough from the face of the concrete to ensure full-width bearing under the ledger: a vertical design load of 900 lb maximum may be used on each dowel.
- For all other cases where timber members bear directly on steel bars, bearing capacity will be verified by means of an "equivalent bearing length" equal to 1/2 the bar diameter. If the calculated stress based on an equivalent bearing length of 1/2 the bar diameter does not exceed the allowable stress, the detail may be authorized even though some crushing will occur.

When a timber member is bearing on a round support, the bearing area obtained by using an equivalent bearing length may not be increased further by applying the effective area factor previously discussed. Combining the two procedures would take unreasonable advantage of the bridging ability of wood fibers, and thus is not permitted for analysis.

5-2.04E Lateral Support of Wood Beams

Beams having a large depth-to-width ratio may fail due to lateral buckling (in much the same manner as long columns) before the allowable bending stress is reached. To avoid this mode of failure the beams must be restrained and forced to deflect in the plane of the load. The amount of restraint needed to ensure beam stability is a function of the depth-to-width ratio, however, it is not subject to precise analysis.

Beam stability must be designed according to NDS 4.4.1, *Stabillity of Bending Members,* using the applicable beam stability factor C_L.

5-2.04F Beam Rollover

In this section, the term beam includes any horizontal load carrying member of the falsework system, including joists. When timber beams are placed in other than a true vertical position, they will have a tendency to rotate about their base as the load is applied. This rotational tendency, commonly referred to as beam rollover, is an indication of instability, which must be investigated during the engineering analysis.

The tendency of beams to roll over when placed on a sloping surface is a function of the height and width of the beam, the load, and the slope on which the beam is placed. Beam rollover should be investigated in all cases where beams are set on a sloping surface. The procedure in Section 5-2.04F(1), *Investigation of Rollover Stability,* describes how to find the limiting beam height for a given load, slope, and beam width.

In addition to rollover stability, beams placed on a sloping surface require a further check to verify that the allowable compressive stress is not exceeded at the down slope corner of the beam.

5-2.04F(1) Investigation of Rollover Stability

For analysis of beam rollover, it is assumed that the vertical load acts as a concentrated load on the top center of the beam. Figure 5-2, *Beam Rollover Forces at Sloping Support,* shows that the load transfers through the beam to the surface in contact with the supporting member through a vertical line. The beam will be stable against rollover if the line of the vertical load reaction lies within the beam width.

When moments are taken about the down slope corner of a beam placed on a sloping surface, as indicated by point A in Figure 5-2, *Beam Rollover Forces at Sloping Support*, the beam will be stable against rollover provided the resisting moment (RM) exceeds the overturning moment (OTM).

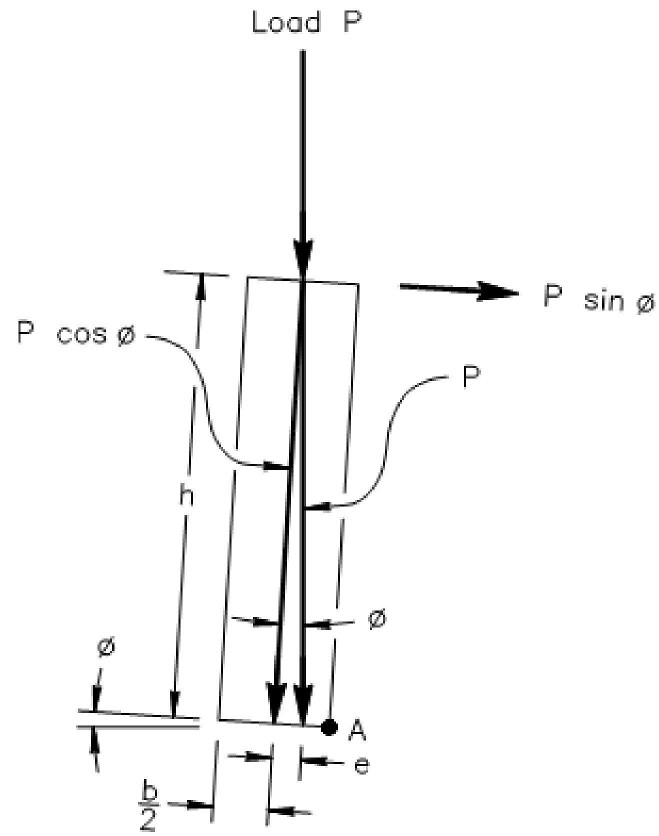


Figure 5-2. Beam Rollover Forces at Sloping Support.

For a given slope and beam load, there is a limiting beam height-to-width relationship. For a given width, the limiting height, **h**, is determined as follows:

$$\mathbf{OTM = RM} \tag{5-2.04F(1)-1}$$

$$\mathbf{h(P)\sin\emptyset = b/2\ (P)\cos\emptyset} \tag{5-2.04F(1)-2}$$

$$\mathbf{(h)tan\emptyset = b/2} \tag{5-2.04F(1)-3}$$

$$\mathbf{h = b/2tan\emptyset \approx b/[2\ (s/100)]} \tag{5-2.04F(1)-4}$$

$$\mathbf{h \approx 50b/s} \tag{5-2.04F(1)-5}$$

where **P** = Load on the beam (lb)

h = Height of the beam (in)

b = Width of the beam (in)

s = Slope (%)

Ø =Tilt angle (deg)

As an example, the limiting slope, **s**, for a 2 x 10 (1.5" x 9.25") beam is calculated as follows:

$$\mathbf{s = 50\frac{b}{h} = \frac{50(1.5")}{9.25"} = 8.1\%} \quad \text{(5-2.04F(1)-6)}$$

5-2.04F(2) Investigation of Compressive Stress

As the slope on which the beam rests increases, the compressive bearing stress between the beam and the supporting member at the down slope edge of the beam increases. This is the case because the center of gravity of the load acting through the top center of the beam remains vertical. The stress at the down slope edge is determined as follows:

- Calculate the compressive stress for normal bearing on the area between the beam and the supporting member as shown in Figure 5-3(a), *Beam Contact Pressure at Sloping Support:*

$$\mathbf{f_{c(a)} = P(cos\emptyset)/A} \quad \text{(5-2.04F(2)-1)}$$

where $\mathbf{f_{c(a)}}$ = Normal compressive stress (psi)

P = Load (lb)

Ø = Slope angle, so that **PcosØ** is the load component acting perpendicular to the bearing surface (deg)

A = Bearing area or contact area (in^2)

- Calculate the stress due to vertical load eccentricity, see Figure 5-3(b), *Beam Contact Pressure at Sloping Support:*

$$\mathbf{f_{c(b)} = Pe(cos\emptyset)/S} \quad \text{(5-2.04F(2)-2)}$$

where $\mathbf{f_{c(b)}}$ = Stress produced by the eccentric loading condition (psi)

e = Distance from the centerline of the beam at the bearing surface to the vertical reaction line (in)

S = Section modulus of the contact area (in^3), see Figure 5-4, *Beam Support Contact Area*

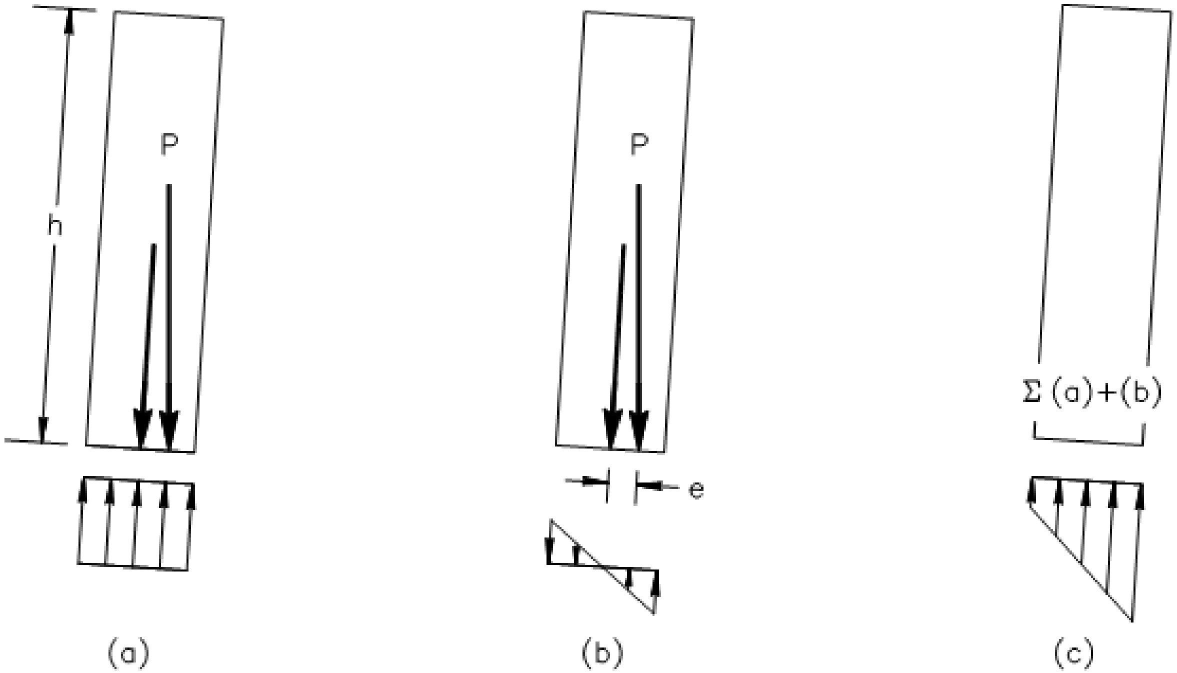


Figure 5-3. Beam Contact Pressure at Sloping Support.

- The sum of the stress values $\mathbf{f_{c(a)}}$ and $\mathbf{f_{c(b)}}$ will give the total compressive stress at the down slope edge of the beam, as shown in Figure 5-3(c), *Beam Contact Pressure at Sloping Support*.

$$\mathbf{f_{c(c)}} = \mathbf{f_{c(a)}} + \mathbf{f_{c(b)}} \qquad \text{(5-2.04F(2)-3)}$$

where $\mathbf{f_{c(a)}}$ = Normal compressive stress (psi)

$\mathbf{f_{c(b)}}$ = Stress produced by the eccentric loading condition (psi)

The calculated bearing stress on the down slope edge of a beam must not exceed the specified allowable bearing stress.

As an example, the bearing stress on the down slope edge of a 2 x 10 (1.5" x 9.25") beam on a 6% cross slope resting on a 1.5 inch wide camber strip where the design load is 500 lb is calculated as follows:

$$\boldsymbol{\varnothing = \tan^{-1}\left(\frac{6}{100}\right) = 3.43^{o}} \qquad \text{(5-2.04F(2)-4)}$$

$$\mathbf{A = 1.5in\ x\ 1.5in = 2.25\ in^2} \qquad \text{(5-2.04F(2)-5)}$$

$$\mathbf{S = \frac{ba^2}{6} = \frac{1.5" \times (1.5")^2}{6} = 0.563\ in^3}$$ (5-2.04F(2)-6)

$$\mathbf{e = h(\tan 3.43^o) = 9.25in(\tan 3.43^o) = 0.554\ in}$$ (5-2.04F(2)-7)

$$\mathbf{f_{c(a)} = \frac{500^{\#}(\cos 3.43^o)}{2.25in^2} = 222\ psi}$$ (5-2.04F(2)-8)

$$\mathbf{f_{c(b)} = \frac{500^{\#}(0.554in)(\cos 3.43^o)}{0.563in^3} = 491\ psi}$$ (5-2.04F(2)-9)

Final stresses is:

$$\mathbf{f_{c(a+b)} = 491\ psi + 222\ psi = 713\ psi < 900\ psi}$$ (5-2.04F(2)-10)

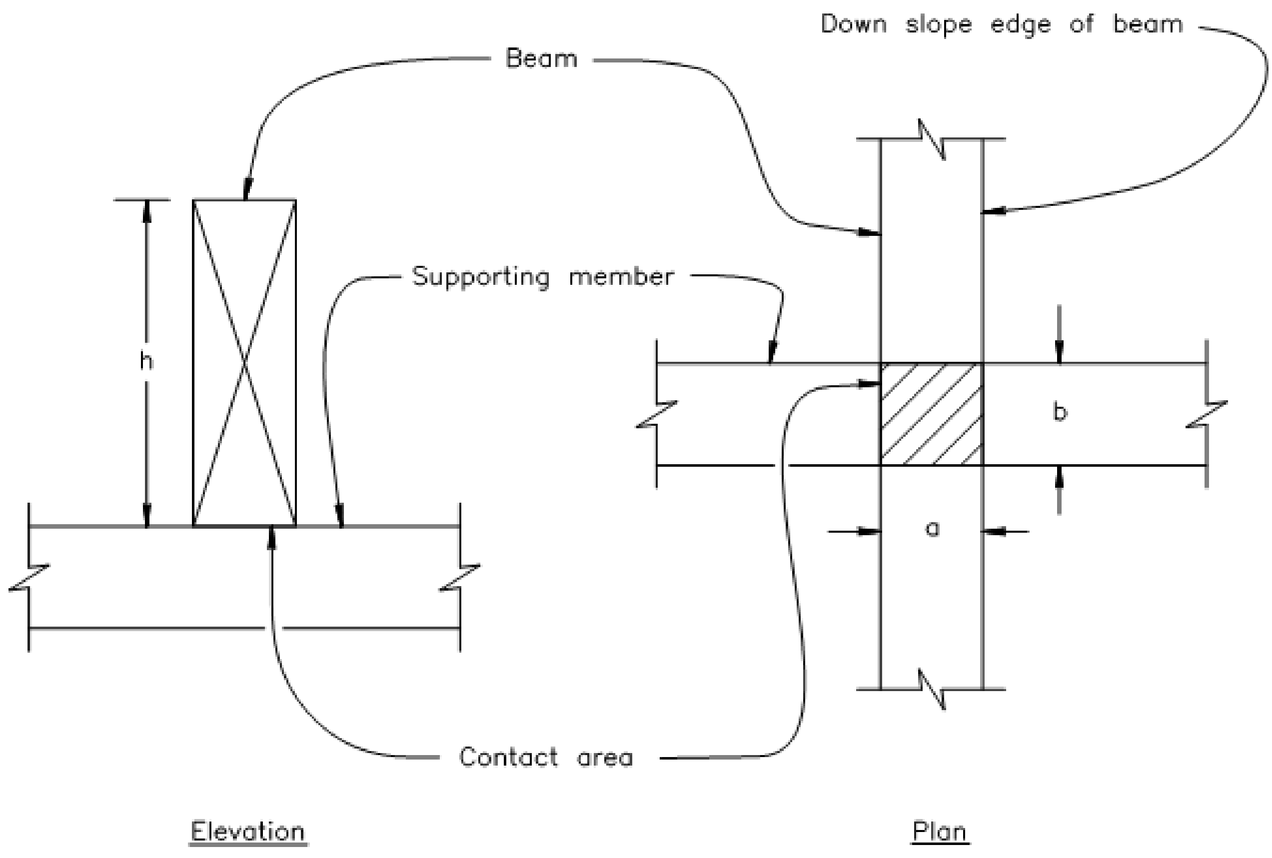


Figure 5-4. Beam Support Contact Area.

5-2.04F(3) Blocking to Prevent Rollover

The tendency of a beam to roll over is an independent condition of instability, therefore, blocking to prevent beam rollover occurs independently of any requirement for blocking or other means of support to ensure beam stability as discussed in Section 5-2.04E, *Lateral Support of Wood Beams*.

Beams which are unstable against rollover when investigated in accordance with the procedures described in sections 5-2.04F, *Beam Rollover,* and 5-2.04F(1), *Investigation of Rollover Stability,* must be made stable by the use of full depth blocking at the beam ends. Additionally, when the slope exceeds 8%, the following apply:

- If the nominal depth-to-width ratio is equal to or less than 4:1, blocking should be provided at mid-span.
- If the nominal depth-to-width ratio is greater than 4:1, blocking should be provided at the 1/3 points of the span.

Toe-nailing to the supporting surface in lieu of blocking will not be permitted for joists that are subject to rollover since the joist can break at the toe-nailed location.

5-2.05 Timber Posts

Timber falsework posts may be considered as pinned at the top and bottom, regardless of the actual end condition, except for driven timber pile bents, which can be considered fixed at a point below ground, see Section 8-6.04, *Timber Piles in Pile Bents*.

Timber posts must be designed in accordance with NDS 3.6, *Compression Members – General.*

The allowable compressive stress, $\mathbf{F_c'}$, is calculated by multiplying the reference design value, $\mathbf{F_c}$, by applicable adjustment factors as shown in NDS Table 4.3.1, *Applicablity of Adjustment Factors for Sawn Lumber*. Reference design values are tabulated in the NDS Supplement.

The combination of compressive stresses parallel to the grain and bending stresses due to buckling are considered in the design of timber posts. To account for buckling, NDS applies a column stability factor, $\mathbf{C_P}$, per NDS 3.7.1, *Column Stability Factor $\mathbf{C_P}$*.

The calculated axial unit stress, $\mathbf{f_c}$, in compression parallel to the grain is determined by dividing the total load by the cross sectional area of the post:

$$\mathbf{f_c = P/A} \tag{5-2.05-1}$$

where $\mathbf{f_c}$ = Compressive stress parallel to the grain (psi)

P = Axial load (lb)

A = Cross sectional area of the post (in^2)

5-2.05A Round and Tapered Posts and Piles

Round and tapered posts and piles shall be designed per NDS 3.7.2 *Tapered Columns*, NDS 3.7.3, *Round Columns,* and NDS Chapter 6, *Round Timber Poles and Piles*.

For round members, the minimum dimension, **d**, should be taken as the length of the side of a square post whose area is equal to the cross sectional area of the round post being used. Round and square posts, having the same cross sectional area, have approximately equal stiffness and therefore carry approximately the same load. This procedure will give results, which are accurate within five percent for posts of the size and length ordinarily encountered in falsework construction.

$$\mathbf{d} = \sqrt{\frac{\pi \mathbf{D^2}}{4}} = \sqrt{\pi \mathbf{R^2}} \tag{5-2.05A-1}$$

where **d** = Side of square post with same cross sectional area as round post with diameter **D** (in)

D = Diameter of round post (in)

R = Radius of round post (in)

5-2.06 Structural Composite Lumber

This section sets forth the practice with respect to the use of structural composite lumber (SCL) as a falsework material.

Except as otherwise provided in this section, all specification requirements and all *Falsework Manual* practice and procedures governing the use of timber members will apply to the use of SCL.

Structural composite lumber is usually marketed commercially as an engineered wood product intended for use as a structural building material and has been used for general building purposes, including limited use in falsework construction. Certain SCL products are manufactured and designed specifically for bridge forming applications.

For inspection and certificate of compliance requirements see Section 9-2.02A, *Structural Composite Lumber*.

5-2.06A General Information

Structural composite lumber is a natural wood product in which the harvested logs are debarked and either peeled or stranded. The resulting veneers or strands are then coated with adhesives and compressed to permanently bond the wood fibers. The finished product is a stronger, straighter and more homogeneous material than conventional lumber.

ASTM D5456 covers test specimen qualification procedures, testing methods and procedures, evaluation of test results, and assignment of design values. The ASTM specification covers composite lumber products which meet the following definitions. SCL intended for structural use is defined as one of the following:

- *Laminated veneer lumber* (LVL): a composite of wood veneer sheet elements with wood fibers primarily oriented along the length of the member. Veneer thickness must not exceed 0.25-inches.
- *Parallel strand lumber* (PSL): a composite of wood strand elements with wood fibers primarily oriented along the length of the member. The least dimension of the strands shall not exceed 0.25-inches, and the average length must be a minimum of 300 times the least dimension.
- *Laminated strand lumber* (LSL): a composite of wood strand elements with wood fibers primarily oriented along the length of the member. The least dimension of the strands must not exceed 0.10-inch, and the average length must be a minimum of 150 times the least dimension.
- *Oriented strand lumber* (OSL): a composite of wood strand elements with wood fibers primarily oriented along the length of the member. The least dimension of the strands must not exceed 0.10-inch, and the average length must be a minimum of 75 times the least dimension.

Typically, NDS dimension lumber sizes, 2 x 4, 4 x 4, 2 x 6, etc., are manufactured from LVL composites, while PSL composites are used for the NDS timber sizes 5 x 5 and larger. LSL and PSL are included as SCL products within ASTM D5456, however, there may not be any manufacturers that market either LSL or PSL for bridge forming applications LSL is typically used as a non-structural edge form material.

Design stress values are a function of grade and wood species, and in some cases the depth and orientation of the member as well. The grade (quality) of a particular lot of material is determined by the modulus of elasticity. Higher modulus values generally correlate with higher allowable design values.

Allowable working stress values are obtained from strength tests on material specimens. Since SCL is a more uniform product than natural wood they can have substantially higher design working stress values than those of even the best grades of lumber. This is largely due to the fact that design values for wood products are based on a characteristic value, which is typically in the lower fifth percentile value. Since strength properties of engineered wood products are more consistent across a population, having a lower coefficient of variation (CoV), the lower fifth percentile value for these products can be substantially higher than for solid sawn lumber, even from the same wood species and timber sourcing region. ASTM adjustment factors from which allowable working stresses are derived are considerably lower for SCL than the corresponding factors for solid sawn wood. These lower adjustment factors result in higher design working stress values for SCL than are allowed for even the best grades of sawn lumber.

The ASTM specification covers procedures for evaluating specific material properties and for determining design values, including bending strength and stiffness, tensile strength parallel to the grain, compressive strength parallel and perpendicular to the grain, and horizontal and vertical shear, along with procedures for maintaining quality assurance in manufacturing. However, the ASTM specification expressly excludes determination of design values for connections.

5-2.06B Design Criteria

The design criteria for SCL is not specifically covered by the *Standard Specifications*, Section 48, *Temporary Structures,* except that the use of SCL is permitted for falsework construction per the *Standard Specifications*, Section 48-2.01C(2), *Shop Drawings,* and as provided herein.

Any intended use of SCL must be indicated by a note on the shop drawings. The note must clearly identify the SCL members by grade (E value), species and type (e.g., 2.0E DF *Trade Name* LVL, or similar notation).

Shop drawings showing the use of SCL must be accompanied by a manufacturer’s published literature and an International Code Council (ICC) report, if available. The technical data shown must include tabulated working stress values for normal load duration and dry service conditions. SCL used for falsework must be manufactured for outdoor use.

The design must be based on working stress using the manufacturer’s tabulated values. The stress values:

- Must not exceed the lesser of the manufacturer's tabulated values or the ICC report.

- Must be adjusted as recommended by the manufacturer for member size or orientation.
- Do not need to be decreased for wet service conditions.

When used as a horizontal load carrying member, the deflection must comply with Section 4-2.01, *Beam Deflection*.

5-3 Timber Fasteners

5-3.01 Introduction

Timber fasteners must be designed per NDS Chapter 11, *Mechanical Connections*. The allowable value in shear, **Z'**, and withdrawal, **W'**, is calculated by multiplying the reference design value, **Z** and **W**, by applicable adjustment factors as shown in NDS Table 11.3.1, *Applicablity of Adjustment Factors for Connections*. Reference design values for dowel type fasteners are determined per NDS 12.2, *Reference Withdrawal Design Values,* and 12.3, *Reference Lateral Design Values*.

Dowel type fasteners are defined as bolts, lag screws, wood screws, nails, spikes, drift bolts, and drift pins.

5-3.02 Connector Design Values

Design values for both lateral load resistance and withdrawal resistance for wood fasteners in various wood species have been standardized by the timber industry, and are available in NDS. Douglas Fir-Larch is commonly used for construction in California.

The design values in NDS are for normal duration of load, and may be increased for the shorter load durations typical of bridge falsework. See Section 5-3.08. *Adjustment for Duration of Load*.

5-3.03 Nails and Spikes

5-3.03A Design Values

The tabulated values are for an individual nail or spike. When more than one nail or spike is used in a connection, the total design value for the connection is the sum of the design values for the individual nails or spikes. This is due to the Group Action Factor, $\mathbf{C_g}$, being equal to 1.0 for dowel type fasteners with a diameter, **D**, less than 1/4-inch, see NDS 11.3.6, *Group Action Factors*.

The tabulated design values are for a nail or spike driven into the side grain of the member holding the point with the axis of the nail or spike perpendicular to the direction of the wood fibers.

Diameters shown in the design tables apply to fasteners before application of any protective coating.

5-3.03A(1) Withdrawal Resistance

Nails and spikes have little resistance to withdrawal when driven into the end grain of a wood member. The use of connections where the nail or spike is subject to withdrawal from the end grain of wood will not be permitted.

5-3.03A(2) Lateral Resistance

The tabulated design values apply to lateral loads acting in any direction.

When a nail or spike is driven into the end grain of a wood member, the design value is multiplied by the end grain factor:

- $\mathbf{C_{eg}}$, per NDS 12.5.2, *End Grain Factor*

The ability of a nail or spike to resist lateral forces is a function of the diameter, **D**, and depth of penetration, **p**, into the member holding the point. The reference design value tables show the maximum lateral design value based on a penetration of 10D into the main member. Penetration beyond 10D does not allow an increase in the reference design value. However, NDS allows for a reduction of design values for penetrations between 10D and 6D. The penetration used in the design must be shown on the shop drawings.

A less than the desired penetration may occur when round posts are used or when longitudinal bracing on skewed bents is not parallel to the side of a square post. In such situations, care must be taken to verify that the minimum penetration is obtained, since nails or spikes having a penetration of less than the minimum will have no allowable lateral load carrying value.

5-3.03B Required Nail Spacing

The timber industry has not adopted standards to govern the spacing of nails and spikes when used in an engineered timber connection. For dowel-type fasteners where the diameter, **D**, is less than 1/4-inch the geometry factor, $\mathbf{C_{\Delta}}$, is equal to 1.0. The guideline in NDS states that “edge distances, end distances, and fastener spacings shall be sufficient to prevent splitting of the wood.”

SC has established the following practice, which will govern the spacing of nails and spikes when used to connect falsework bracing components:

- The average center-to-center distance between adjacent nails or spikes, measured in any direction, must not be less than the required penetration into the main member for the size of nail being used.
- The minimum end distance and the minimum edge distance in both side member and main member, must not be less than 1/2 of the required penetration.

While proper installation of timber connections is primarily a field concern, the design review must assure that the members are large enough to accommodate the required number of nails or spikes when they are spaced in conformance with the criteria listed above.

5-3.03C Toe-Nailed Connections

National Design Specifications recommends that toe-nails be driven at an angle of approximately 30° to the member being toe-nailed, and started approximately 1/3 of the nail length, from the end of the member. The penetration of the nail should be a minimum of 6 times the diameter (**6d**). See Figure 5-5, *Toe-Nailed Connection*.

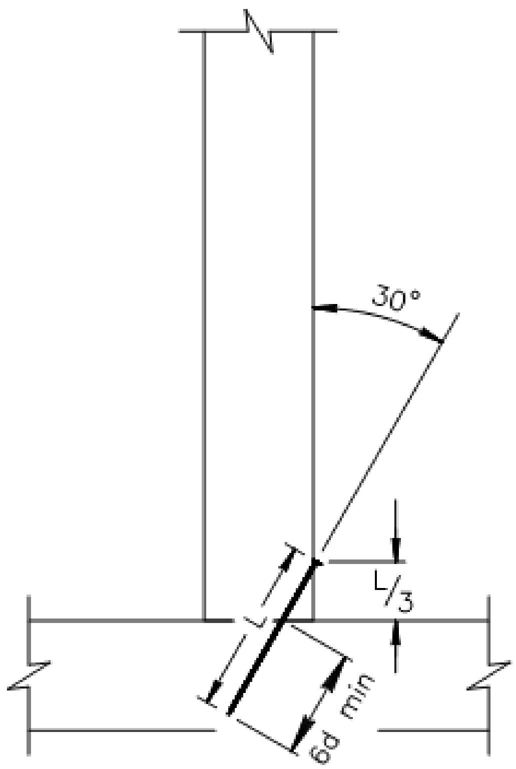


Figure 5-5. Toe-Nailed Connection.

Design values for withdrawal and lateral load resistance must be multiplied by the toe-nail factor, C_{tn}, for toe-nailed connections as follows:

- C_{tn} = 0.67 for withdrawal loading
- C_{tn} = 0.83 for lateral loading

5-3.04 Bolted Connections

5-3.04A Design Values

The tabulated design values for bolted connections are provided in the NDS.

Threaded rods and coil rods may be used in place of bolts of the same diameter with no reduction in the tabulated values.

For connections in falsework, adjacent to or over railroads, substitution of bolts with coil rods or threaded rods is permitted if the root diameter is equal to the shank of the required bolt diameter.

If the connection is loaded at an angle to the bolt axis (e.g. a longitudinal brace on a skewed bent) see NDS section 12.3.10, *Load at an Angle to Fastener Axis*.

When the load is applied at an angle to the grain, as is the case with falsework bracing, the design value for the main member is obtained from the Hankinson formula.

$$\mathbf{N} = \frac{\mathbf{PQ}}{\mathbf{P sin^2 \theta + Q cos^2 \theta}} \tag{5-3.04A-1}$$

where: **N** = Design value for the main member (lb)

P = Tabulated design value for a load applied parallel to grain (lb)

Q = Tabulated design value for a load applied perpendicular to the grain (lb)

θ = Angle between the direction of the wood grain in the main member and the direction of the load in the side member (deg)

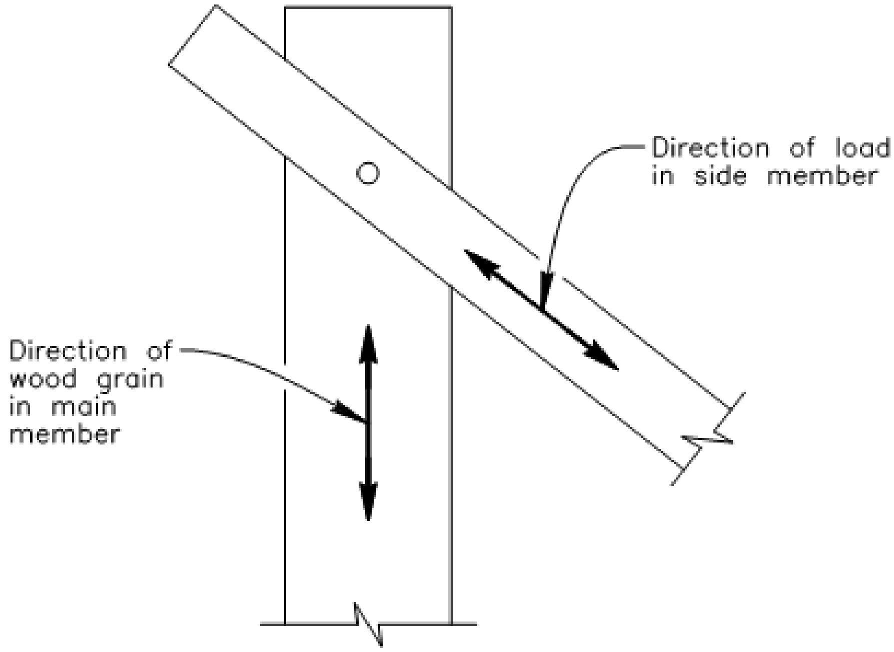


Figure 5-6. Wood Grain Direction in Post and Brace.

The reference values in NDS, are based on square posts. For round posts, the main member thickness is taken as the side of a square post having the same cross-sectional area as the round post.

$$\mathbf{d} = \sqrt{\frac{\pi \mathbf{D}^2}{4}} = \sqrt{\pi} \mathbf{R}^2 \quad \text{(5-3.04A-1)}$$

where **d** = Length of a square post having the same cross sectional area as a round post with diameter D (in)

D = Diameter of round post (in)

R = Radius of round post (in)

5-3.04B Design Procedure for Wood Cross Bracing

Special requirements apply to connections of diagonal wood bracing in compression. Referring to Section 6-3.02, *Wood Cross Bracing*:

- The contribution of the compression members and compression member connections is limited to 1/2 of their theoretical vales as calculated in the following sections.

5-3.04C Single Shear Connections

Figure 5-7, *Single Shear Bolted Connection* shows a typical two-member bolted connection in which the side member is loaded parallel-to-grain and the main member is loaded at an angle to the grain. NDS provides reference design values for single shear connections for both wood and steel side members.

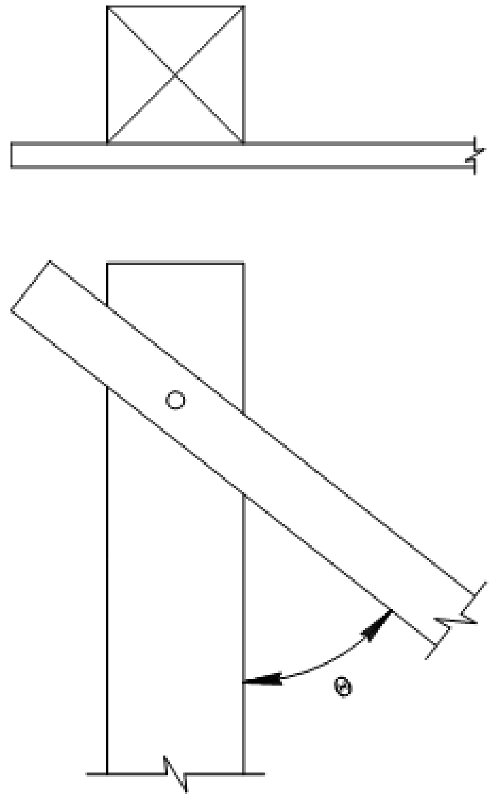


Figure 5-7. Single Shear Bolted Connection.

5-3.04D Double Shear Connections

Figure 5-8, *Double Shear Bolted Connection,* shows a three-member bolted connection in which the side members are loaded parallel-to-grain and the main member is loaded at an angle to the grain. NDS provides reference design values for double shear connections for both wood and steel side members.

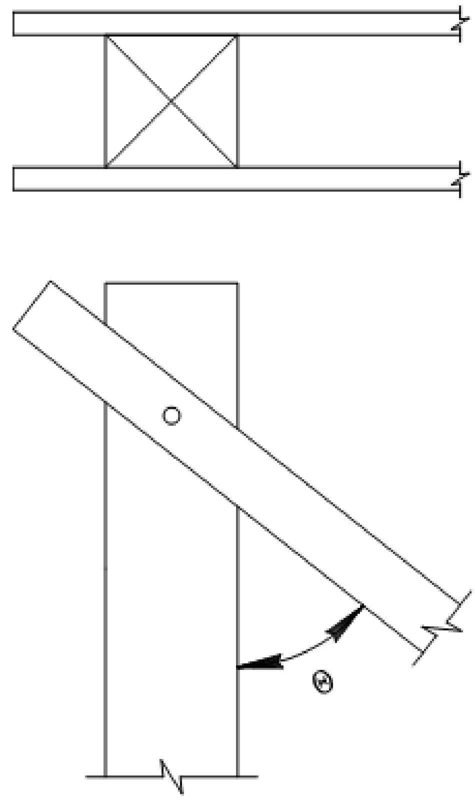


Figure 5-8. Double Shear Bolted Connection.

5-3.04E Installation Requirements

Although installation is primarily a construction concern, the design review should verify that installation of bolts meets the NDS criteria for end and edge distance found in NDS section 12.5, *Adjustment of Reference Design Values*.

For multiple bolt connections see section 5-3.06, *Multiple Fastener Connections.*

5-3.05 Lag Screw Connections

The design procedure for lag screws is similar to bolts, see section 5-3.04, *Bolted Connections*. Reference design values for lag screws for both withdrawal and lateral loading may be found in the NDS.

The reference design values apply only when the lag screw is installed in a properly sized predrilled hole.

5-3.06 Multiple Fastener Connections

Multiple fastener connections are designed in accordance with the NDS by applying the group action factor, $\mathbf{C_g}$,:

- C_g per NDS 11.3.6, *Group Action Factors*

5-3.07 Drift Pin and Drift Bolt Connections

Drift pins are steel rods cut to any desired length. Drift bolts are steel rods manufactured with a bolt head on one end. Typically, these fasteners are used to connect large members, such as caps and posts, in a timber bent. Drift pins and drift bolts are designed in accordance with NDS.

5-3.08 Adjustment for Duration of Load

Design values shown in the NDS are for normal load durations and may be increased for short duration loading.

Selecting the proper load duration factor to use in the calculations is a matter of engineering judgment. Typical load duration factors, $\mathbf{C_D}$, can be found in NDS Table 2.3.2, *Frequently Used Load Duration Factors*. For typical falsework design, a 7-day load duration factor is commonly used. For loads of shorter duration, such as wind, a larger factor would be appropriate. For stage construction where the falsework will remain loaded for an appreciable length of time, a lower factor may be appropriate.

The load duration factors, $\mathbf{C_D}$, for normal falsework construction are:

- C_D = 1.25 for vertical and horizontal loads
- C_D = 1.60 for wind load
- C_D = 2.0 for impact loading (limited to 1.6 for connection and pressure treated lumber)

If the falsework will remain loaded for a relatively longer period of time, the use of smaller duration of load factors is appropriate. These situations occur during cast-in-place prestressed construction where stressing will be delayed or during stage construction sequences for any type of concrete structure.

5-4 Steel Members

5-4.01 Design Criteria

The Standard *Specifications*, Section 48-2.02B(3)(c), *Stresses, Loading, and Deflections – Steel,* allows the use of the current *American Institute of Steel Construction (AISC) Manual* for design of steel except for flexural compressive stresses, deflections, and modulus of elasticity.

The specifications permit the use of grades of steel higher than ASTM Grade A36 for all loading conditions, provided the grade of steel can be identified. Identification is the contractor's responsibility, subject to verification by the engineer, see Section 9-2.03, *Steel Members*.

Design of unidentified steel is based on the assumed use of structural steel conforming to ASTM Grade A36.

Falsework over or adjacent to railroad must also comply with the current railroad guidelines.

5-4.02 Allowable Stresses

The maximum allowable stresses for identified steel must not exceed the requirements in the current *AISC Manual*, except for flexural compressive stresses, which must not exceed the requirements in *Standard Specifications*, Section 48-2.02B(3)(c), *Stresses, Loadings, and Deflections – Steel*.

The maximum allowable stresses for unidentified steel are based on the assumed use of structural steel conforming to ASTM Grade A36 and must not exceed the requirements in the current AISC Manual or those in the *Standard Specifications*, except for flexural compressive stresses, which must not exceed the requirements in *Standard Specifications*, Section 48-2.02B(3)(c), *Steel*.

Although the specifications allow higher working stresses when higher strength steel is used, the load carrying capacity of steel beams will, in most cases, be limited by deflection, not stress. When deflection controls, the use of high strength steel will not be of any benefit since the limiting deflection cannot be increased.

High strength steel beams may provide a greater load carrying capacity in situations where beams are subjected to bi-axial bending.

5-4.03 Bending Stresses

The bending stress formulas are:

$$\mathbf{f} = \frac{\mathbf{Mc}}{\mathbf{I}} = \frac{\mathbf{M}}{\mathbf{S}} \qquad (5\text{-}4.03\text{-}1)$$

where **f** = Bending stress (psi)

M = Bending moment (in-lb)

c = Distance from the neutral axis to the extreme fiber (in)

I = Moment of inertia of the beam about the neutral axis (in^4)

S = Beam section modulus (in^3)

If the compression flange is supported, these formulas may be used to determine the section needed to carry the applied load for a beam in which bending occurs in the vertical plane only. However, bridge falsework differs from most other construction in that top caps are typically set to the slope of the bridge soffit rather than level and the stringers are set flush on the cap. This construction configuration causes the stringers to deviate from a true vertical plane by an angle which is equal to the soffit cross slope, and the result is bi-axial bending in the stringer. Bi-axial bending is discussed in Section 5-4.04, *Bi-Axial Bending*.

If the compression flange of a beam is not supported, the maximum allowable bending stress must be reduced to prevent flange buckling, see Section 5-4.05, *Flange Buckling*.

5-4.04 Bi-Axial Bending

Figure 5-9, *Steel Beam on a Sloping Support,* shows a beam set on a sloping surface (i.e. canted). For such beams, the theoretical centroid of the applied load, **P**, acts on the top flange through the projected centerline of the web, rather than through the center of gravity of the canted beam.

When a vertical load is applied to a canted beam, the load is divided into two components: one acting through the web, and one acting along the top flange. This loading condition produces bi-axial bending (i.e. bending in two planes) which decreases the beam capacity. The decrease in beam capacity is a function of beam shape and soffit cross slope, and it cannot be determined by inspection. The effect of bi-axial bending must be investigated in all cases where falsework beams are not set in a true vertical plane.

When a beam is set on a sloping surface, the load component acting along the top flange causes the flange to deflect in the down slope direction. For nominal cross slopes, this lateral deflection is small and may be neglected. As the cross slope increases, however, the lateral deflection increases as well, and eventually becomes a factor for consideration since it can adversely affect both form alignment and reinforcing steel clearances.

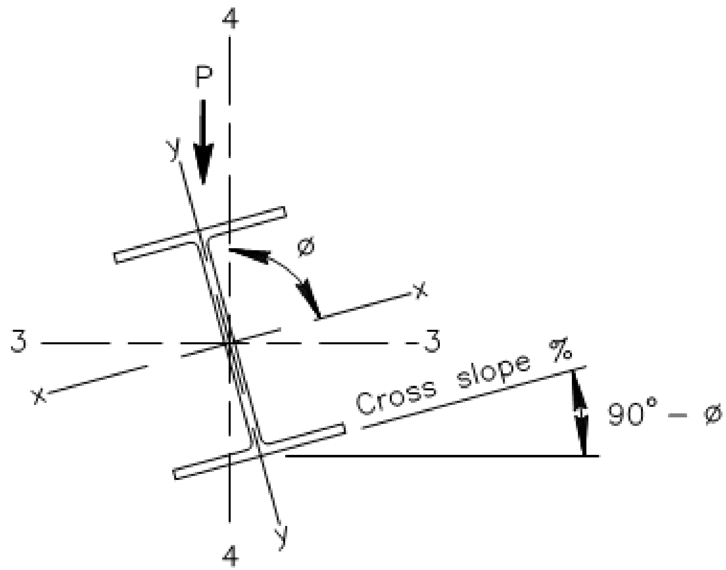


Figure 5-9. Steel Beam on a Sloping Support.

For analysis of bi-axial bending, lateral deflection must be considered in all cases where the falsework beam is canted more than 2%.

5-4.04A Beams Canted 2% or Less

Refer to Figure 5-9, *Steel Beam on a Sloping Support*. For any beam subject to bi-axial bending, the maximum bending stress, $\mathbf{f_b}$, is the sum of the bending stresses in the **x** and **y** directions. The following formulas may be used to calculate bending stress, $\mathbf{f_b}$, based on the moments of inertia of the non-rotated section and the rotation angle, **Ø**.

$$\mathbf{f_b} = \mathbf{M}\left[\frac{\mathbf{y}}{\mathbf{I_{xx}}}\mathbf{sin}\,\emptyset + \frac{\mathbf{x}}{\mathbf{I_{yy}}}\mathbf{cos}\,\emptyset\right] \quad (5\text{-}4.04A\text{-}1)$$

$$\mathbf{I_3} = \mathbf{I_{xx}}\mathbf{sin^2}\emptyset + \mathbf{I_{yy}}\mathbf{cos^2}\emptyset \quad (5\text{-}4.04A\text{-}2)$$

$$\mathbf{I_4} = \mathbf{I_{xx}}\mathbf{cos^2}\emptyset + \mathbf{I_{yy}}\mathbf{sin^2}\emptyset \quad (5\text{-}4.04A\text{-}3)$$

where **M** = Applied moment (in-lb)

y = Distance from the x-axis to the extreme fiber (in)

x = Distance from the y-axis to the extreme fiber (in)

Δ = Deflection angle (deg)

$\mathbf{I_{xx}}$ = Moment of inertia about x-axis of the beam (in^4)

I_{yy} = Moment of inertia about y-axis of the beam (in^4)

I_3 = Moment of inertia about the horizontal axis (in^4)

I_4 = Moment of inertia about the vertical axis (in^4)

Calculate the actual vertical deflection by using the moment of inertia about the 3-axis, **I_3**, in the deflection equation. As previously noted, for this case the lateral deflection may be neglected.

As an alternative procedure, stresses and deflections may be determined with respect to the strong x-axis and weak y-axis by using the **x** and **y** components of the applied load, **P**.

5-4.04B Beams Canted More Than 2%

The maximum bending stress and vertical deflection are computed in accordance with the procedure for beams canted 2% or less as discussed in the preceding section. In addition, for box girder structures, it is necessary to evaluate the effect of lateral deflection as discussed herein.

For box girder structures, the net lateral deflection of falsework beams under the weight of the bottom slab and girder stems only is limited to 1.5 inches. This limitation is considered necessary to mitigate the adverse effect of lateral movement during the soffit and girder stem concrete pour on form alignment and reinforcing steel placement and clearances.

Refer to Figure 5-10, *Lateral Deflection of a Canted Beam*, which is a schematic depiction of the movement of a point (Point A) on the top flange of a beam which is subject to bi-axial bending as it deflects under load.

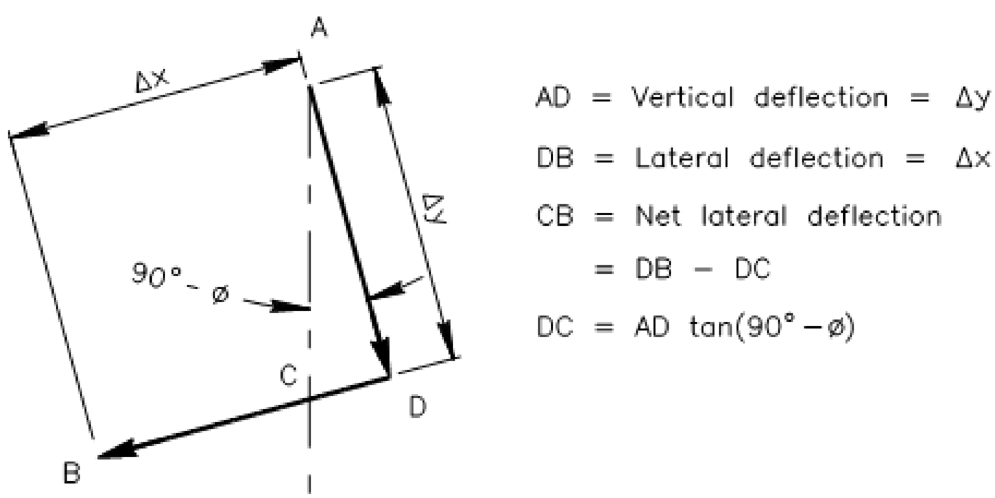


Figure 5-10. Lateral Deflection of a Canted Beam.

The movement of point **A** to point **B** is the combined vertical and lateral deflection of the bottom slab and stems of a box girder structure as the concrete is placed. While the vertical deflection can be compensated for by camber strips, the lateral deflection **DB** will displace the bottom slab and stems from the planned alignment, line **AC**, by the distance **CB**, which is the net lateral deflection. The net deflection is limited to 1.5 inches maximum.

For the lateral deflection calculation, the vertical load is the dead load weight of the concrete in the soffit slab and girder stems. Use the component values of the vertical load to determine beam deflections about both the x-axis and the y-axis.

5-4.05 Flange Buckling

The compression flange of a beam may be visualized as a column loaded along its length by increments of load transferred to it by horizontal shear from the web. If the compression flange is narrow in comparison to the depth of the beam, the flange may fail by buckling in a similar manner as a slender column.

Although methods for determining the critical buckling stress are quite complex, in steel members many simplifications are possible. Generally, the empirical formulas used are similar to column formulas, except that the flange width, **b**, is used instead of the radius of gyration.

The *Standard Specifications*, Section 48-2.02B(3)(c) *Steel* limit the allowable compression stress from flexure in the beam to:

$$\mathbf{f} = \frac{\mathbf{12{,}000{,}000}}{\mathbf{Ld/bt}} \qquad \text{(5-4.05-1)}$$

where **f** = Maximum allowable compressive stress due to flexure (psi)

L = Laterally unsupported length (in)

d = Depth of the member (in)

b = Width of compression flange (in)

t = Thickness of the compression flange (in)

The maximum allowable stress is limited to:

- 22,000 psi for unidentified steel
- 22,000 psi for steel conforming to ASTM designation A36
- 0.6 $\mathbf{F_y}$ for other identified grades of steel where $\mathbf{F_y}$ is the minimum yield stress.

If the actual stress exceeds the allowable, the flange must be supported or the load reduced.

It is difficult to determine how much lateral support may be developed by other falsework members. For example, friction between the joists and top flange of a beam will provide some restraint, but the amount is indeterminate. Therefore, friction between the joists and top flange will be neglected when investigating flange buckling.

Since it is impossible to predict the direction in which the compression flange will buckle, it is necessary to provide positive restraint in both directions. This is an important and often overlooked point. For example, the use of a tension tie between two adjacent beams or welding a light structural steel shape (such as a bar or angle) across the top flange of several beams will not prevent lateral movement. Such measures merely assure that all beams deflect in the same direction. Even when tension ties are used in combination with a compression strut, lateral restraint is not effective because the restraining components cannot assure that the beams will act as a unit when a lateral force is applied.

Timber cross bracing between adjacent beams is widely used for flange support in falsework construction. In this method timber struts, in pairs, are set diagonally between the top flange of one beam and the bottom flange of the adjacent beam to form an “X”, and securely wedged into place. However, timber cross bracing alone will not prevent flange buckling because timber struts can resist only compression forces.

Perhaps the most effective flange support method is to use steel tension ties, banded, welded, clamped, or otherwise secured across the top and bottom of adjacent beams in combination with timber cross bracing between the beams, as shown in Figure 5-11, *Two-Stringer Cross Bracing*.

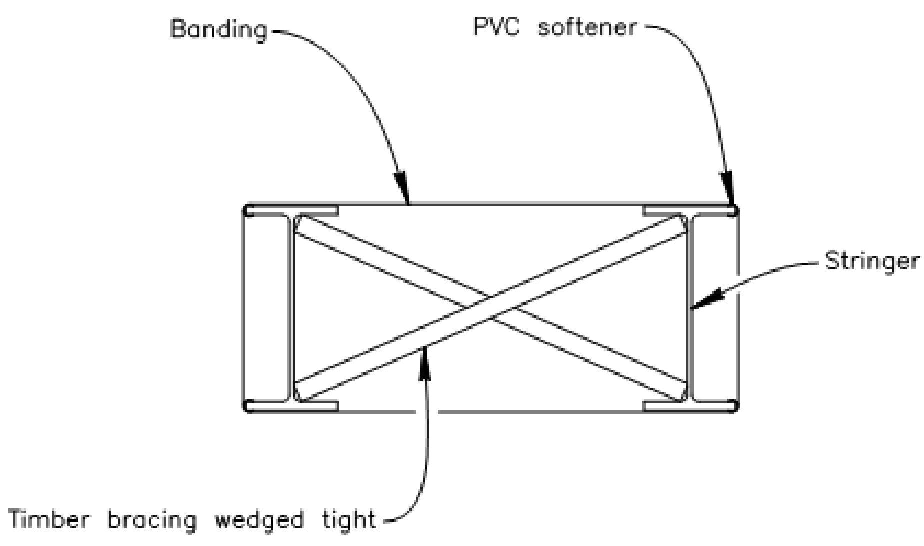


Figure 5-11. Two-Stringer Cross Bracing.

Commercial steel banding material wrapped around pairs of adjacent cross braced beams is commonly used. Steel banding is less expensive and easier to install and remove than other types of tension components. However, banding is not effective unless it is properly installed and tightened. When banding is used as part of a flange support system, some means must be provided to prevent an abrupt bend or kink at the point of contact with the outer edge of the beam flange. This is an important consideration because any kink or sharp bend in commercial banding is, potentially, a point of stress concentration, which can reduce the strength of the banding material. The use of softeners will reduce this stress concentration, see Figure 5-11, *Two-Stringer Cross Bracing*. If there is any question as to the adequacy of banding installed in a given situation, the contractor should be required to furnish the manufacturer's catalog data and instructions.

When rebar is welded to the top flange as tension ties, as a minimum, bottom tension ties must be installed in the end bays. The bottom tension ties can also be rebar welded to the bottom flanges. See Figure 5-12, *Multi-Stringer Cross Bracing*.

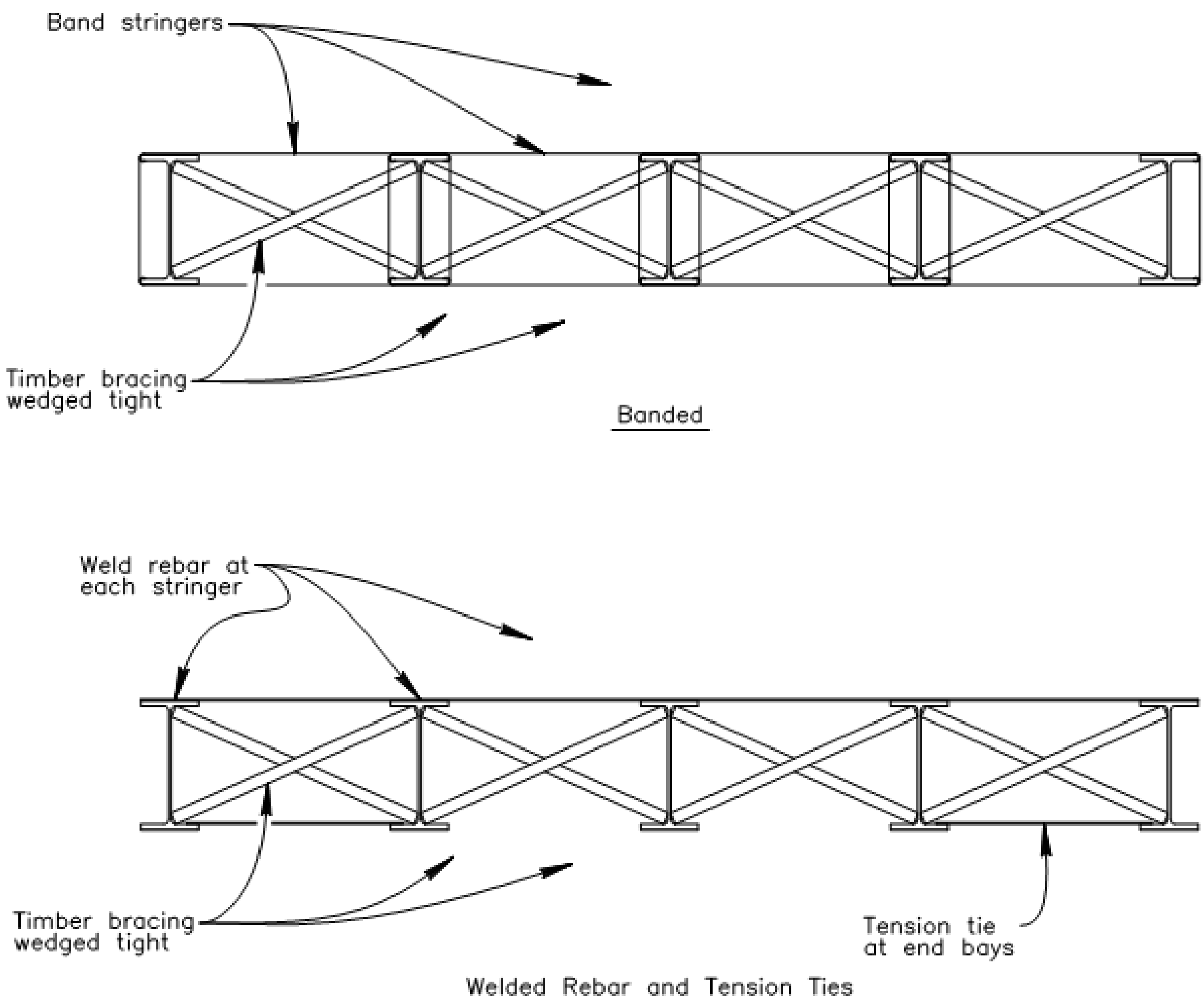


Figure 5-12. Multi-Stringer Cross Bracing.

Bracing, blocking, steel banding, ties, etc., required for lateral support of beam flanges must be installed at right angles to the beam. Bracing in adjacent bays should be set in the same transverse plane, if possible. If, because of skew or other considerations, it is necessary to offset the bracing in adjacent bays, the offset distance should not exceed twice the depth of the beam.

Only a small force is needed to hold the compression flange in position. In steel design for permanent work, it is common practice to use an assumed value of 2% of the calculated compression force in the beam flange at the point under consideration as the design force for the supporting brace. This practice is also acceptable for bridge falsework. The point under consideration should be designed for the maximum moment for the segment supported by the brace.

Providing adequate flange support is an important design consideration when support is necessary to prevent overstressing the compression flange. The method of support, including all construction details, must be shown on the shop drawings.

5-4.06 Beam Shear

The shearing stress at any point in a steel beam is calculated from the general formula for shearing stress. The general formula is:

$$\mathbf{f_v = \frac{VQ}{Ib}} \qquad (5\text{-}4.06\text{-}1)$$

where $\mathbf{f_v}$ = Shearing stress on any horizontal section or plane through the beam (psi)

V = Vertical shear (lb)

Q = Static moment about the neutral axis of that portion of the beam cross section, which is outside of the plane where the shearing stress is wanted (in-lb)

I = Moment of inertia of the entire beam cross section about the neutral axis (in^4)

b = Width of the beam at the point under consideration (in)

Since the web of a steel beam resists most of the shear, the shearing stress is usually calculated by:

$$\mathbf{f_v = \frac{V}{ht}} \qquad (5\text{-}4.06\text{-}2)$$

where f_v = Shearing stress through the web (psi)

V = Vertical shear (lb)

h = Overall depth of the beam including flanges (in)

t = Web thickness (in)

If a shearing stress occurs in one plane, an equal shearing stress occurs in a plane through that point perpendicular to the first plane. Therefore, the shear formula may be used to determine both vertical and horizontal shearing stresses.

5-4.07 Web Yielding

Rolled beams should be checked to verify the end reaction and any concentrated load along the interior of the beam does not produce a compressive stress at the web toe of the fillet, in excess of the allowable stress. If the actual value exceeds the allowable, the web must be stiffened or the length of bearing increased. When rolled beams require bearing stiffeners to prevent web yielding, the stiffeners may be designed to resist only the portion of the total load that is in excess of the load the beam is capable of resisting without stiffeners.

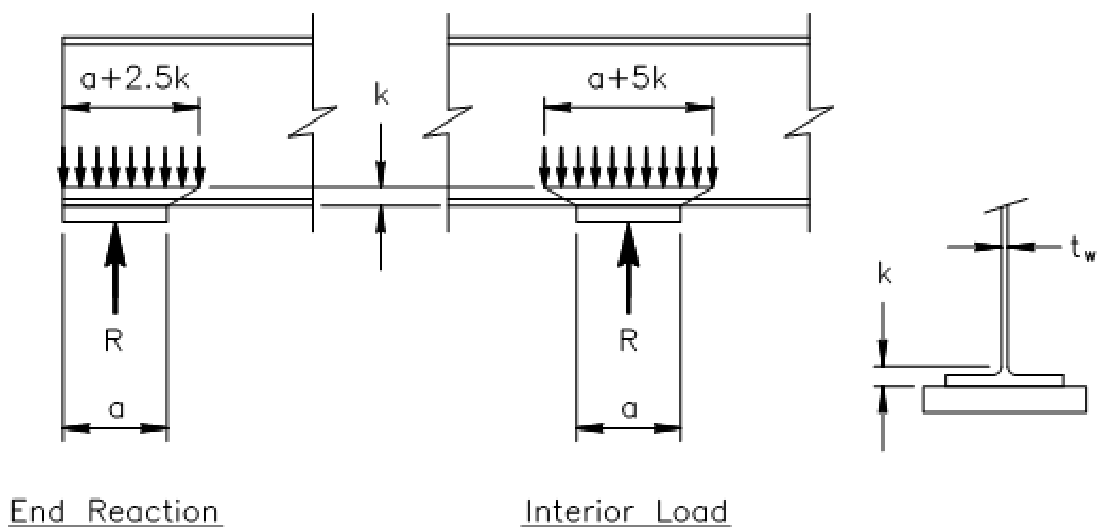


Figure 5-13. Web Yielding.

5-4.08 Web Crippling

Web crippling should be checked in beams with slender webs. Web crippling may occur at locations of concentrated loads and at supports. Figure 5-14, *Web Crippling,* illustrates the behavior of the web when it cripples.

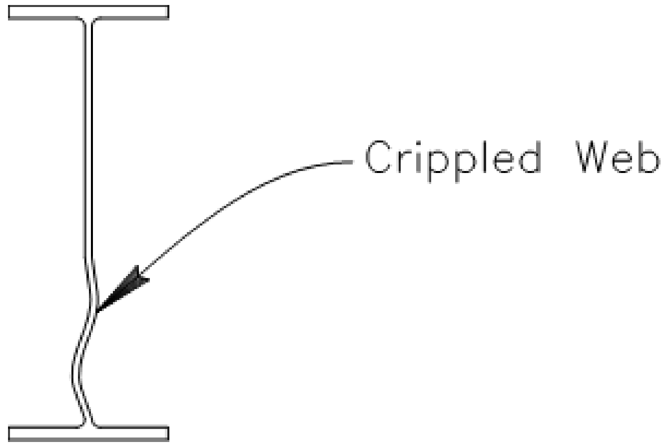


Figure 5-14. Web Crippling.

5-4.09 Flange Bending

Flange bending should be checked for steel to timber connections at locations of concentrated loads and supports. Figure 5-15, *Localized Flange Bending,* illustrates the behavior of the flange when it bends over the web. The flange capacity is determined by:

$$\frac{R_n}{\Omega} = \beta_1 t_f^2 F_{yf} \tag{5-4.09-1}$$

where R_n = Flange capacity (lb)

β_1 = Constant based on uniform stress distribution

t_f = Flange thickness (in)

F_{yf} = Minimum specified yield strees of the flange (psi)

Ω = 1.67 (Factor of Safety)

Table 5-1. β_1 values for beams assuming uniform stress distribution.

Section	HP 12x53	HP 14x73	HP 14x89	W 14x90	HP 14x117	W 14x120
β_1	10.9	13.1	13.6	16.2	14.9	18.1

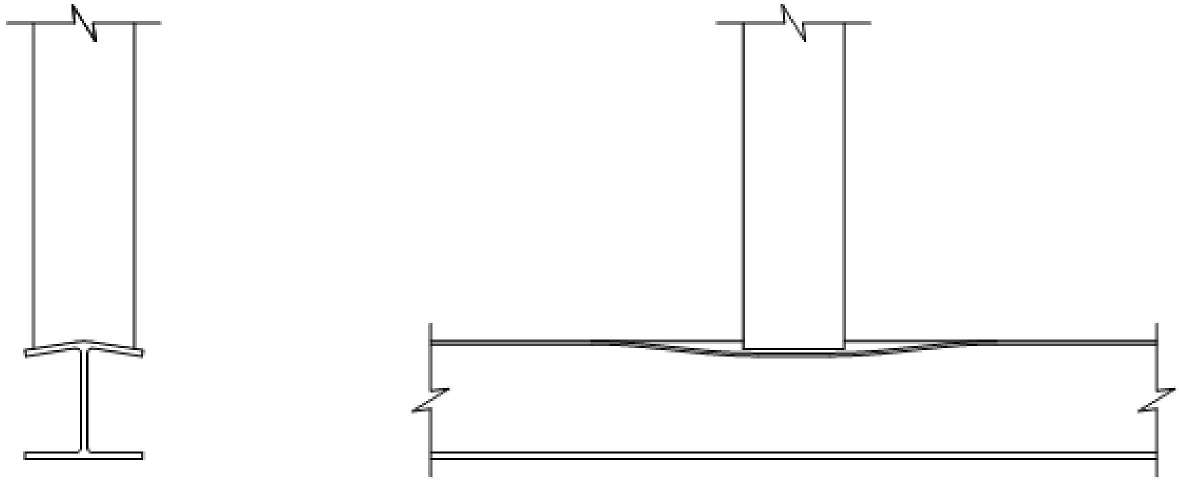


Figure 5-15. Localized Flange Bending.

5-4.10 Lateral Web Buckling

Buckling of unbraced, unstiffened beams, where the flange is loaded with a post load has potential to displace sideways through buckling of the web and is synonymous with column buckling. The dimensions of the assumed column are as follows:

- Column height equal to the clear distance between the beam flanges
- Column depth equal to the web thickness
- Column width equal to the tributary width of the associated post, which is typically the post spacing for interior post

Analyse using elastic buckling formula found in the *AISC Steel Construction Manual* with a effective length factor equal to 1.7.

5-4.11 Timber Blocking

Timber blocking can be used to increase capacity for web yielding, web crippling, and flange bending. Timber blocking must not be used for web lateral buckling. The full capacity of the blocking is not effective for increasing web capacity. The effective capacity is given in the following formula:

$$\mathbf{P_b = \gamma F_{c||}'A_b} \qquad (5\text{-}4.11\text{-}1)$$

γ = 0.5 for wood post

γ = 0.3 for steel post

where $\mathbf{P_b}$ = Capacity of timber blocking (lb)

γ = Blocking effectiveness factor

$\mathbf{F_{c\|}'}$ = Nominal allowable stress for block after adjustment factors are applied (psi)

$\mathbf{A_b}$ = Combined cross sectional area of blocking on both sides (in^2)

Location of blocking is limited to the locations shown in Figure 5-16, *Timber Blocking*.

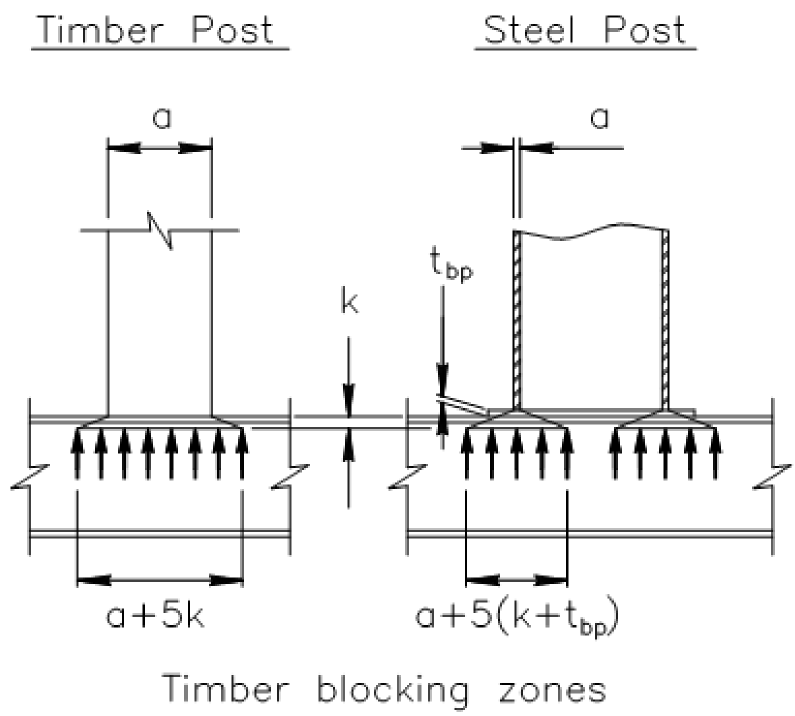


Figure 5-16. Timber Blocking

5-4.12 Steel Posts

In a post with pinned ends and no intermediate support, the unsupported length, **L**, is the actual length. Posts with other end conditions require the use of an effective length instead of the actual length. The effective length is the length of post, which actually behaves as though it were pinned.

Determining the effective length of a post with restrained end conditions is an unnecessary refinement for falsework design. It is accepted practice to treat posts in falsework bents as though their ends are pin-connected, which is conservative for typical falsework posts with some end restraint.

5-4.13 Steel Bracing

For bolted steel connections, the bolt design values may be taken from the AISC *Steel Construction Manual*. In accordance with AISC design practice, the calculated bearing stress on the projected area of the bolt in steel members may not exceed 1.35 times the specified yield strength, F_y, of the steel. For A36 Grade material, the allowable bearing stress is 48,600 psi. This value may not be increased for falsework construction.

Structural steel components (angles, bars, etc.) are sometimes used as diagonal bracing in timber bents. In such cases, as discussed in Section 5-3.04A, *Design Values,* the bolt design values for parallel-to-the-grain loading in the main member may be increased, but no increase is allowed for perpendicular-to-the-grain loading.

5-4.14 Welding Steel Members

Refer to *Standard Specifications*, Section 48-2.01D(2), *Welding and Nondestructive Testing,* for welding requirements. Refer to Section 9-2.03B, *Welding Steel Members,* for inspection requirements. *Standard Specifications*, Section 11, *Welding,* does not apply to welding of falsework members.

5-4.14A All Welds

All welding must comply with AWS D1.1 welding standard. All welds must be performed by a certified welder and inspected by an independent qualified inspector as stated in AWS D1.1.

The design strength must be determined in accordance with the AISC design practice. Per the *Standard Specifications*, Section 48-2.01C(2), *Shop Drawings*, the welding standard must be shown on the shop drawings.

5-4.14B Welded Splices

Special requirements apply to welded splices. The contractor is required to follow the requirements in the *Standard Specifications*, Section 48-2.01D(2), *Welding and Nondestructive Testing,* when splicing steel members by welding. See Section 9-2.03, *Steel Members,* for additional details and inspection requirements.

5-4.14C Approximating Fillet Welds

The design strength of fillet welded connections may be approximated by assuming a value of 1000 lb per inch length for each 1/8-inch of the fillet weld size (e.g. 1-inch length of a 1/4-inch weld has a strength of 2000 lb). This value is considered conservative for work performed by a certified welder and therefore is only intended as a rough approximation. If the capacity is not adequate using this approximation, the method in the AISC Steel Construction Manual should be used to determine the capacity of fillet welds.

5-4.14D Grades Higher Than A36

If the falsework design is based on the higher working stresses allowed for grades of steel other than Grade A36, the contractor must furnish substantiating mill test reports and a Certificate of Compliance. The Certificate of Compliance must be signed by the contractor with a list and description of the beams covered by the mill test reports.

5-4.15 Miscellaneous Steel Components

The adequacy of miscellaneous components (such as anchor bolts, post base plates, grillages, hangers, tie bars, and similar steel components) and hardware items, when used at locations subject to analysis but not specifically covered by the *Falsework Manual*, will be determined in accordance with applicable design procedure or recommended practice included in the AISC Steel Construction Manual.

5-5 Cable Bracing Systems

5-5.01 Introduction

The term cable bracing system means a length of wire rope cable and an appropriate fastening device. Cable bracing systems may be used to resist both overturning and collapsing forces, See Chapter 6, *Stability,* for a discussion of overturning and collapse as falsework failure modes.

Cable systems are particularly effective in resisting the overturning of high falsework. They are also commonly used as diagonal bracing to resist collapse of falsework bents. Moreover, cable is also used extensively as temporary bracing to stabilize falsework bents while they are being erected and/or removed. Cable systems are relatively inexpensive compared to other bracing methods.

Cables and cable fastening devices are an essential part of the falsework design. Design of cable systems is a sophisticated exercise, particularly with respect to such factors as the anticipated cable elongation, the amount of preloading or cable tension needed, the effect of cable tension on other members, and similar factors, which affect system stability.

SC practice with respect to the use of cable bracing systems, and the procedures and methodology to be used by field engineers when reviewing the adequacy of cable designs, are discussed in the following sections.

The guidance provided herein can be used when prestressed (PT) strands are used for bracing. However, it is worth noting that PT strands have a modulus of elasticity, **E** value, in the range of 28,000 - 29,000 ksi as compared to cables that have an **E** value of about 13,500 ksi. In addition, the PT strand has yield values as 90% of minimum breaking force as compared to falsework cables that has yield strength as 65% of the minimum breaking force.

As used in this section, the term *cable* includes prestressing strand when prestressing strand is used in a falsework bracing system.

5-5.02 Required Information

All elements of the cable bracing system must be shown on the shop drawings in sufficient detail to permit a stress analysis. In addition, the contractor must provide technical data showing the strength and physical properties of the cable to be used, see Section 5-5.03, *Manufacturer's Technical Data*.

The following information is to be shown on the shop drawings for all cable systems:

- Cable diameter
- Cable description (including cable nominal diameter, number of strands, number of wires per strand, and core type)
- Weight of the cable
- Minimum breaking force
- Net metallic area
- Modulus of elasticity
- Maximum construction stretch (percent of loaded length)
- Preload value, along with the method by which the preload force is to be measured
- The type and number of fastening devices (such as Crosby clips, plate clamps, turnbuckles, shackles, etc.) to be used at each connection.
- If tightening is necessary, the method by which the cables may be tightened after installation to ensure their continued effectiveness
- Cable anchorage
- The location and method of attachment of the cable to the falsework

The location and method of attachment of the cable to the falsework are of particular importance to the design, since the connecting device must transfer both horizontal and vertical forces to the cable without overstressing any falsework component. When cables are used to prevent overturning of heavy duty shoring, cable restraint must be designed to act through the cap system. Cables should not be attached to tower components unless the contractor has obtained written authorization from the shoring system manufacturer. Such authorization must be furnished before the shop drawings are authorized. (See Chapter 7, *Manufactured Assemblies.*)

5-5.03 Manufacturer's Technical Data

For the application of the *Standard Specifications*, Section 48-2.02B(3)(d), *Manufactured Assemblies*, a cable bracing system (i.e., the cable together with cable fastening devices) is a manufactured assembly. Therefore, the cable must be installed in accordance with the manufacturer's recommendations, and the contractor must furnish manufacturer's technical data if requested by the engineer.

For all cable systems, technical data from the manufacturer must include:

- Cable diameter
- Cable description (including cable nominal diameter, number of strands, number of wires per strand, and core type)
- Weight of the cable
- Minimum breaking force (which may be identified as a rated or nominal strength) or the safe working load of the cable
- Net metallic area
- Modulus of elasticity
- Maximum construction stretch (percent of loaded length)
- Preload value
- The type and number of fastening devices (such as Crosby clips, plate clamps, turnbuckles, shackles, etc.) to be used at each connection.

Since the adequacy of a cable bracing design cannot be verified without reference to the technical data provided by the cable manufacturer, such data is an essential part of the shop drawing submittal. If the shop drawings include a cable bracing system and are not accompanied by technical data from the cable manufacturer, the contractor should be informed immediately. The shop drawing submittal is not complete until the technical data is provided, see Chapter 2, *Review of Shop Drawings*.

In the absence of technical data, a load test will be required to establish cable strength and physical properties, see Section 5-5.10, *Cable Load Tests*.

5-5.04 Cable Connector Design

Cable connectors must be designed in accordance with criteria shown in Table 5-2, *Wire Rope Connections,* and Table 5-3, *Number and Spacing of U-Bolt Wire Rope*

Clips. Connector efficiency (CE) assumed in the design must not exceed the values shown in Table 5-2, *Wire Rope Connections*.

Connector efficiency factor for PT strands does not apply when used with chuck/wedges/retainer cap.

The installation of cable connectors must conform to the manufacturer's requirements. Only forged clips must be used as connectors. Forged clips are marked *forged* to permit positive identification, and have the appearance of galvanized metal. Malleable clips shall not be used as connectors. Malleable cable clips appear smooth and shiny.

If U-bolt (Crosby type) wire rope clips are used as connectors, the number used and the spacing must conform to the data shown in Table 5-3, *Number and Spacing of U-Bolt Wire Rope Clips*, and must be shown on the shop drawings.

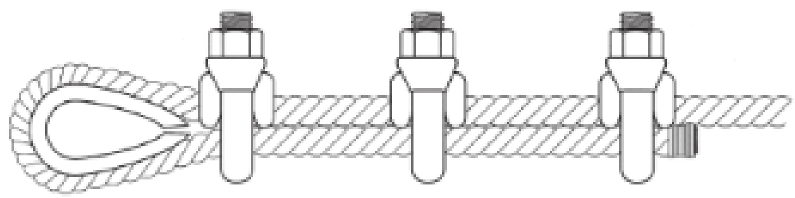


Figure 5-17. Applying Wire Rope Clips.

The only correct method of attaching U-bolt wire rope clips to rope ends is shown in Figure 5-17, *Applying Wire Rope Clips*. The base (saddle) of the clip bears against the live end of the rope, while the "U" of the bolt presses against the dead end. A useful method of remembering this is: "You never saddle a dead horse."

The clips are usually spaced about six rope diameters apart to give adequate holding power. A wire rope thimble should be used in the loop eye to prevent kinking when wire rope clips are used. The correct number of clips for safe application, and spacing distances are shown in Table 5-3, *Number and Spacing of U-Bolt Wire Rope Clips*.

Table 5-2. Wire Rope Connections.

Wire Rope Connections (As compared to Safe Load on Wire Rope)		
Figure	**Type of Connection**	**Efficiency**
	Wire Rope	100 %
2	Sockets – Zinc Type	100 %
3	Wedge Sockets	70 %
4	Clips – Crosby Type	80 %
5	Knot and Clip (Contractor's Knot)	50 %
6	Plate Clamp – Three Bolt Type	80 %
7	Spliced Eye and Thimble	
	1/4" and smaller	100 %
	3/8" to 3/4"	96 %
	7/8" to 1"	88 %
	1-1/8" to 1-1/2"	82 %
	1-5/8" to 2"	75 %
	2-1/8" and larger	70 %

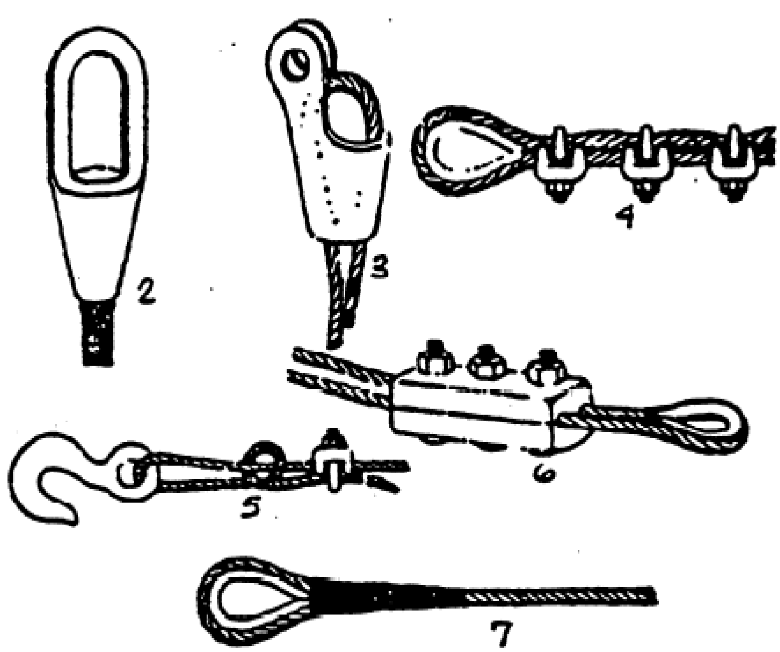


Table 5-3. Number and Spacing of U-Bolt Wire Rope Clips.

U-Bolt Wire Rope Clips		
Improved Plow Steel, Wire Rope Diameter (in)	Number of Clips (Drop Forged)	Minimum Spacing (in)
1/2	3	3
5/8	3	3-3/4
3/4	4	4-1/2
7/8	4	5-1/4
1	5	6
1-1/8	6	6-3/4
1-1/4	6	7-1/2
1-3/8	7	8-1/4
1-1/2	7	9

5-5.05 Cable Elongation

Wire rope cable is an elastic material, which will elongate or stretch when loaded. However, cable is a unique elastic material in that its elongation is not uniform throughout the elastic range. Additionally, it is subject to inelastic deformation at loads well below the yield strength.

For descriptive purposes, the cable industry identifies the two properties that contribute to the total elongation experienced by a cable during its service life as elastic stretch and construction stretch.

To ensure falsework stability, cable elongation must be considered when cable is used as bracing to prevent overturning or collapse of the falsework.

5-5.06 Factor of Safety

The allowable (or design) load carrying capacity of a product or device is obtained by applying a factor of safety based on the ultimate strength of that product or device. In general, this approach is satisfactory, because the system integrity will not be jeopardized by inelastic deformation that may occur if a product or device is subjected to a load that exceeds its yield strength, provided the load is not greater than the ultimate strength. However, this practice is not appropriate for cables used as falsework bracing because of the need to limit the total cable elongation to a predictable amount. In view of this reality, when cable is used as falsework bracing, the allowable working load must be related to the yield strength of the cable.

While a factor of safety of two, based on yield strength, may be considered satisfactory for falsework, the industry standards established by the Wire Rope Technical Board require the safe working load for static loading conditions to be determined using a factor of safety of three, based on the minimum breaking force of the cable. Since cables of this type used for falsework have yield strength of approximately 65% of the nominal strength, the industry standard is consistent with the use of a factor of safety of two based on yield strength.

Therefore, a factor of safety, **FS = 3**, based on the minimum breaking force, **MBF**, is required when determining the allowable design capacity of the cable units.

5-5.07 Limitations and Conditions of Use

The use of cable bracing systems designed to resist collapse is limited to single tier falsework bents where the cable is fastened to the bent cap. The use of cable to provide frame rigidity in multi-tiered bents, or in any bent where the cable is attached to a post or column will not be permitted, see Chapter *6, Stability*, for definitions of single and multi-tiered bents.

For single tier falsework with cable bracing systems, the horizontal deflection is limited to:

$$\boldsymbol{\Delta = \frac{L}{8} \leq \frac{b}{4}} \qquad \text{(5-5.07-1)}$$

where Δ = Horizontal deflection (in)

L = Post height (ft)

b = Post width or diameter (in)

Limiting the horizontal deflection is necessary to prevent undesirable frame distortion, and to ensure that the vertical load reaction remains within the base width of the post.

The calculated horizontal deflection must be based on cable elongation due to both elastic and construction stretch, as discussed later in this section.

Cables attached to timber members, must be attached with an appropriate fastening device installed in accordance with the manufacturer's recommendations. Looping cable bracing around timber members is not permitted, because of the need to accurately predict the amount of lateral deflection in the system. However, for temporary erection or removal of bracing it is acceptable to loop cable bracing around timber members.

5-5.08 Cable Design Load for Overturning

When cables are used as bracing to resist overturning, the horizontal design load to be resisted by the cable bracing system will be calculated as follows:

- For heavy duty shoring, cable bracing must be designed to resist the difference between the total overturning moment and the resistance to overturning provided by the individual towers, see Chapter 4, *Design Considerations,* and Chapter 6, *Stability,* for overturning considerations.
- For pipe-frame shoring, cable bracing must be designed to resist the difference between the total overturning moment and the resistance to overturning provided by the shoring system as a whole, see Chapter 7, *Manufactured Assemblies,* for analysis of pipe frame shoring systems.
- For all other types of falsework, including temporary bracing used to stabilize falsework components during erection and/or removal, cable bracing must be designed to resist the total overturning moment.

5-5.09 Review Criteria for Cable Bracing Systems

The procedure for analyzing cable bracing systems, assumes that the post is attached to the top cap and bottom cap by an eccentric pinned connection, and that the eccentricity is numerically equal to the horizontal movement of the top cap due to cable unit elongation. These assumptions are valid for typical pipe post bents where the connections are not designed to resist moment, and for all timber bents. However, it is theoretically possible to design a pipe post frame with fixed connections. Any such designs will require a rigorous analysis by the contractor, with supporting calculations, and similar review by the engineer. In such cases, the engineer should contact the Falsework Engineer in SC HQ for the procedure to be followed.

The procedure considers the effect of cable elongation and frame deflection. Cable elongation allows the frame to deflect, producing vertical load eccentricity, which must be considered in the analysis. Additionally, post reactions will be affected by the vertical component of the cable load.

5-5.09A Allowable Working Loads

The maximum allowable load to be carried by a given cable will be determined in accordance with the following criteria:

5-5.09A(1) New Cable

If the cable is new or in uniformly good condition, and can be identified by reference to a manufacturer's catalog or similar technical publication, the allowable cable load will be the minimum breaking force, **MBF**, of the cable as shown in the manufacturer's catalog, multiplied by the efficiency of the cable connector, and divided by the factor of safety. The allowable load capacity is:

$$\mathbf{P} = \frac{(\mathbf{MBF})(\mathbf{CE})}{\mathbf{FS}} \qquad \text{(5-5.09A(1)-1)}$$

where **P** = Allowable cable load capacity (lb)

MBF = Minimum breaking force (lb)

CE = Connector efficiency, see Table 5-2, *Wire Rope Connections*

FS = Factor of safety

While the technical data provided by the manufacturer will, in most cases, show the minimum breaking force, of the cable; some manufacturer's catalogs show only a recommended safe working load. If this is the case, the cable design load may not exceed the safe working load value, unless the contractor elects to perform a load test.

For a given cable, the safe working load recommended by the manufacturer is considerably less than the allowable load determined from the minimum breaking force of the cable. This is the case because the recommended safe working load is based on a factor of safety of 5 or more in consideration of the dynamic loading to which cable is ordinarily subjected. However, the appropriate static loading condition associated with falsework construction is the factor of safety of 3.

5-5.09A(2) Used Cable

Used cables must be in serviceable condition. A cable in serviceable condition will pass the inspection of a competent person and will comply with all the requirements for rope inspection in the current edition of the *Wire Rope User's Manual*, published by the Wire Rope Technical Board. For inspection, see Section 9-3.12D(2), *Cable Inspection*.

If the cable is used and still in serviceable condition, but a manufacturer's catalog is not available, the contractor may elect to perform a load test. In such case, the allowable working load must not exceed the minimum breaking force, **MBF**, as determined by the load test, multiplied by the connector efficiency factor, and divided by the factor of safety. The allowable load capacity is:

$$\mathbf{P} = \frac{(\mathbf{MBF})(\mathbf{CE})}{\mathbf{FS}} \qquad \text{(5-5.09A(2)-1)}$$

where **P** = Allowable cable load capacity (lb)

M**BF** = Minimum breaking force (lb)

CE = Connector efficiency, see Table 5-2, *Wire Rope Connections*

FS = Factor of safety

If the cable is used and still in serviceable condition, and if the contractor does not perform a load test, the allowable working load must not exceed the wire rope capacity shown in Table 5-4, *Wire Rope Capacities,* multiplied by the cable connector efficiency. The allowable load capacity is:

$$\mathbf{P} = (\mathbf{SL})(\mathbf{CE}) \qquad \text{(5-5.09A(2)-2)}$$

where **P** = Allowable cable load capacity (lb)

SL = Safe load from Table 5-4, *Wire Rope Capacities* (lb)

CE = Connector efficiency, see Table 5-2, *Wire Rope Connections*

Table 5-4. Wire Rope Capacities.

Wire Rope Capacities		
Safe load for new improved plow steel hoisting rope		
6 strands of 19 wires. Hemp Center		
FS = 6		
Diameter	**Weight**	**Safe Load**
(in)	(plf)	(lb)
1/4	0.10	1,050
5/16	0.16	1,500
3/8	0.23	2,250
7/16	0.31	3,070
1/2	0.40	4,030
9/16	0.61	4,840
5/8	0.63	6,330
1	1.60	15,000
1-1/8	2.03	18,600
1-1/4	2.50	23,000
1-3/8	3.03	26,900
1-1/2	3.60	30,700
1-5/8	4.23	36,700
1-3/4	4.90	41,300

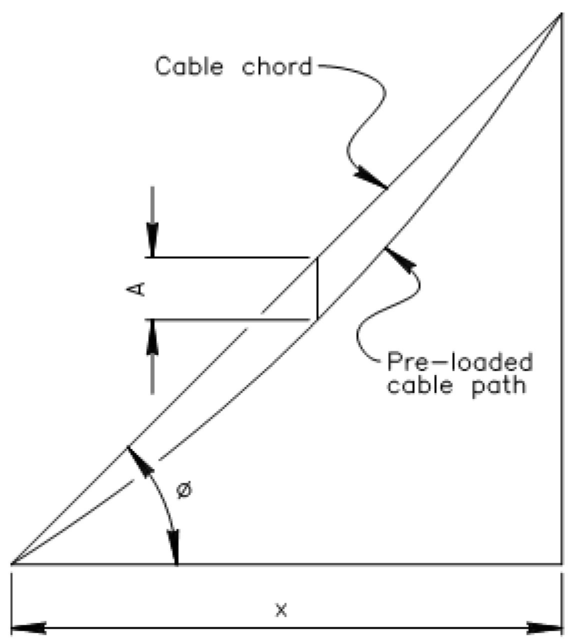
5-5.09B Cable Preload

In this section, the term *cable unit* refers to all cables acting to resist forces in the same direction, and the term *cable* refers to each individual cable within the cable unit.

After assembly, all cable units must be preloaded to remove any slack in the cables and connections. Preloading is necessary to ensure that the cable units will act elastically when the loads are applied.

A number of subjective considerations are involved when determining whether a given preload force is sufficient to ensure that the cable bracing system will act elastically. In the past, an arbitrary value of 1000 lb has been commonly used as a minimum preload force; however, the actual force needed to remove cable slack is a function of both the length and weight of the cable, thus there is no single preload value that will be appropriate for all installations.

To verify design adequacy, use the relationship between preload force and the theoretical drape of the cable hanging under its own weight. For any given preload value, the drape may be calculated using the formula in Figure 5-18, *Cable Drape Formula*. Dimension **A** is the mid-span cable drape.



$$A = \frac{wx^2}{8T\cos\phi}$$

Where A = Deflection (ft)
w = Cable weight per foot (lb/ft)
x = Horizontal distance between cable connection points (ft)
T = Cable pre-load value (lb)

Figure 5-18. Cable Drape Formula.

The preload force shown on the shop drawings must tension the cable unit sufficiently so that the calculated cable drape, after the falsework is erected, will not exceed the values in Table 5-5, *Maximum Cable Drape*.

Table 5-5, Maximum Cable Drape.

Cable Diameter	Max Cable Drape "A"
(in)	(in)
3/8	1
1/2	2
5/8	2-3/4

Experience has demonstrated that a preload force of less than about 500 lb may be insufficient to remove all cable slack. Therefore, the minimum preload value must not be less than 500 lb, regardless of other considerations.

For bents where the top cap and bottom cap do not have the same slope, the variation in post height will produce a non-symmetrical cable arrangement wherein the opposing cable units will have different lengths, and thus different elongations under a given preload force. However, the horizontal component of the cable elongation at the top cap connection must be equal in both directions to prevent distortion of the falsework bent. This means that, except for symmetrical cable arrangements, opposing cable units will have slightly different preload values.

Preload values are to be calculated by the contractor and shown on the shop drawings, along with the method by which the required preload force is to be measured.

5-5.09C Cable Elongation

The assumptions and design practice discussed herein are based on recommendations and design standards in the *Wire Rope User's Manual,* Fourth Edition, 2005, published by the Wire Rope Technical Board. Industry recommendations and standards are modified as appropriate for falsework considerations.

For a given installation and design load, the total cable elongation is a function of two independent factors:

- Elastic stretch, which is the result of the inherent elasticity, or recoverable deformation, of the metal itself. Since the elastic properties of a given cable can be determined, elongation due to elastic stretch is predictable.
- Construction stretch, which occurs when cable is loaded for the first time. When a cable is first loaded, the helically wound wire and strands are pulled more tightly together, compressing the core and bringing all of the cable elements into closer contact. This results in a slight reduction in diameter and a corresponding increase in length. Construction stretch is influenced by several factors including:
    - Type of core
    - Number of strands

- Number of wires in each strand
- The manner in which the cable is wound
- The magnitude of the applied load

Because of the number of variable factors involved, there is no mathematical constant applicable to all cable types from which elongation due to construction stretch may be determined. For a given cable and load, however, the probable construction stretch can be approximated with sufficient accuracy for cable design considerations.

5-5.09C(1) Elastic Stretch

For an elastic material loaded within the elastic range, the elastic deformation (i.e., the change in length, or stretch) is directly proportional to the change in applied load, all other factors remaining equal.

The general formula for elastic deformation is:

$$\Delta = \frac{(\textbf{Change in Load})(\textbf{Length})}{(\textbf{Area})(\textbf{Modulus of Elasticity})} \qquad \text{(5-5.09C(1)-1)}$$

Unlike other elastic materials, cable elongation is not directly proportional to the applied load over the full elastic range. This is the case because the modulus of elasticity for a given cable is significantly lower at low levels of applied load than at loads nearer to the normal working strength of the cable.

To accommodate this unique physical characteristic, it is standard practice in the cable industry to facilitate cable elongation calculations by using a nominal **E** value and a reduced **E** value, depending on the magnitude of the applied load. The nominal **E** value is used for that portion of the total load, which exceeds 20% of the minimum breaking force of the cable. The reduced **E** value, which is equal to 90% of the nominal value, is used for the portion of the load between zero and 20% of the minimum breaking force.

If the cable design load is not greater than 20% of the cable minimum breaking force , the elastic stretch may be determined from the general formula for elastic deformation shown above, using the reduced **E** value:

$$\Delta = \frac{(\mathbf{P-Preload})(\mathbf{L})}{\mathbf{A(0.9E)}} \qquad \text{(5-5.09C(1)-2)}$$

where Δ = elastic deformation (ft)

P = Cable load (lb)

L = Loaded length of the cable (ft)

A = Net metallic area of the cable (in^2)

E = Nominal modulus of elasticity (psi)

If the cable design load is greater than 20% of the minimum breaking force, the total elastic stretch is the sum of Δ_1 and Δ_2 as given by the following formulas:

$$\Delta_1 = \frac{(0.2MBF - Preload)(L)}{A(0.9E)} \tag{5-5.09C(1)-3}$$

$$\Delta_2 = \frac{(P - 0.2MBF)(L + \Delta_1)}{AE} \tag{5-5.09C(1)-4}$$

$$\Delta_T = \Delta_1 + \Delta_2 \tag{5-5.09C(1)-5}$$

where Δ_1 = Elastic deformation below 20% of minimum breaking force (ft)

Δ_2 = Elastic deformation above 20% of minimum breaking force (ft)

Δ_T = Total elastic deformation (ft)

MBF = Minimum breaking force (lb)

L = Loaded length of the cable (ft)

P = Cable load (lb)

A = Net metallic area of the cable (in^2)

E = Nominal modulus of elasticity (psi)

5-5.09C(2) Construction Stretch

As previously noted, construction stretch occurs when a cable is loaded for the first time. Construction stretch is an important design consideration for cable bracing systems because, depending on cable type, a typical new wire rope cable initially loaded to its design working strength will undergo a permanent elongation of from 1/2 to 1% of the loaded length.

Industry design practice assumes that construction stretch is proportional to the applied load, and that all construction stretch occurs within the elastic range. That is, the total expected construction stretch will have occurred when the applied load reaches the yield point load, or 65% of the cable minimum breaking force.

Construction stretch is given by:

$$\Delta_{\mathbf{CS}} = \left(\frac{\mathbf{P}}{\mathbf{0.65MBF}}\right)(\mathbf{CS})(\mathbf{L}) \qquad \text{(5-5.09C(2)-1)}$$

where $\Delta_{\mathbf{CS}}$ **=** Construction stretch (ft)

P = Applied load (lb)

CS = Anticipated construction stretch provided by manufacturer

L = Cable length between end connections (ft)

MBF = Minimum breaking force (lb)

Construction stretch is expressed as a percent of the loaded length of the cable. For falsework bents, the loaded length is the length between end connections. The anticipated construction stretch, will be included in the cable design data provided by the manufacturer. If for some reason it is not provided and cannot be obtained, the analysis may be based on assumed values of 3/4 and 1% for wire core and fiber core cables, respectively.

Some types of high strength cable, such as prestressing strand, are commercially available with construction stretch removed by preloading at the factory. Such cable will conform to the requirements for ASTM Designation A586 (structural strand) or ASTM Designation A603 (structural rope), and will be clearly identified as prestretched cable. When prestretched cable is used, it is not necessary to consider construction stretch in the analysis.

Cables conforming to ASTM Designation A586 or A603 may be either prestretched or non-prestretched. Prestretched cable must be so identified in the cable design data furnished by the manufacturer. If the cable is not clearly identified as prestretched, construction stretch must be considered in the analysis, even though the cable may otherwise conform to the referenced ASTM specifications.

5-5.09D Horizontal Displacement of Top Cap

When calculating the horizontal displacement of the top cap due to cable unit elongation, all posts are assumed to rotate about their bases, and their tops move laterally the same distance as the cap. The calculated horizontal displacement must be less than the allowable horizontal displacement. See Section 5-5.07, *Limitations and Conditions of Use*.

Calculate horizontal displacement:

- Refer to Figure 5-19, *Post Displacement due to Cable Elongation*.
- The vertical distance between the lower cap and upper cap cable connection points at the location of the cap cable connection, vertical line **a**, may be used to complete triangles for the preloaded **b** and fully loaded **b'** cables. The law of cosines may be used to determine angles, since the dimensions of all three legs of the triangles will be known. Once the angle of rotation, **Ø**, of the posts has been determined, the horizontal displacement at the tops of the posts can be calculated. The angle **α** is the slope of the cap.

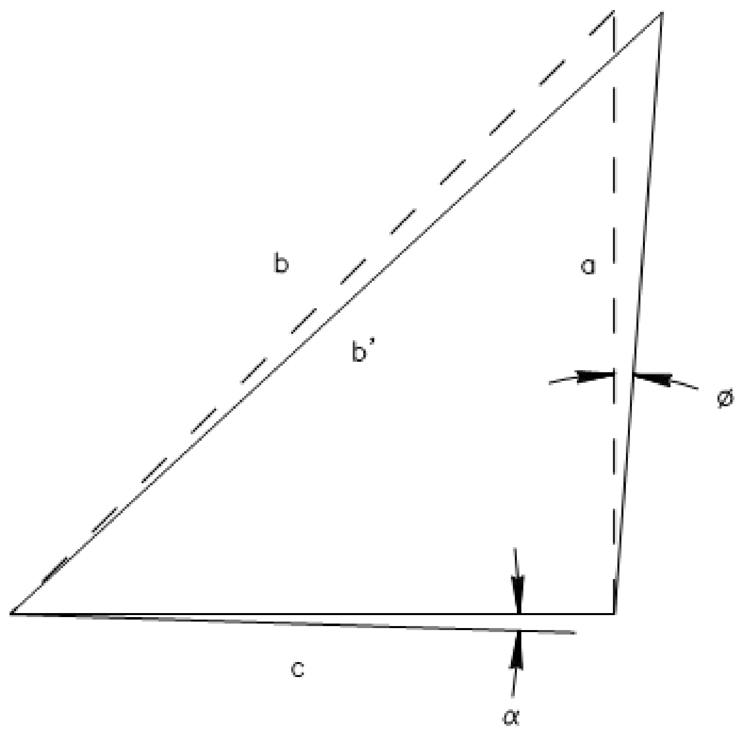


Figure 5-19. Post Displacement due to Cable Elongation.

5-5.09E Step By Step Procedure for Analysis

The following procedure is used to evaluate the system. This procedure is illustrated in Appendix D *Example Problems*, Example 18, *Cable Bracing – Bents*:

- Determine cable lengths, post heights, and the vertical distance between the plane of the cable connection at the lower cap and the plane of the cable connection at the upper cap.
- Calculate the horizontal design load based on the assumed horizontal load, see Section 3-3, *Horizontal Load*, and *Standard Specifications*, Section 48-2.02B(2), *Design Criteria – Loads*. The assumed horizontal load is applied to the unloaded and the loaded conditions.

- Calculate the capacity of the cable units, using the factor of safety based on the minimum breaking force, see Section 5-5.06, *Factor of Safety*.
- Check the cable preload values shown on the shop drawings.
- Using the horizontal design load, calculate the cable unit design load.
- Compare the cable unit design load and the cable unit capacity. If the design load exceeds capacity, the system must be redesigned.
- Calculate the cable unit elongation, which is the sum of the elongations due to elastic and construction stretch. As previously noted, a consideration of both elastic and construction stretch is required when calculating the expected cable elongation, unless the cable to be used has been preloaded at the factory to remove the construction stretch.
    - Calculate the elastic stretch
    - Calculate the construction stretch
    - Add the elastic stretch and construction stretch to obtain the total elongation for the cable unit
- Calculate the horizontal displacement of the top cap due to cable unit elongation
- Compare the calculated horizontal displacement and the allowable horizontal displacement

5-5.09E(1) Final Steps for Box Girder Bridges

The *Standard Specifications*, Section 48-2.02B(2), *Design Criteria – Loads,* provide for two loading conditions for box girder structures with cable bracing:

- Live load, stem and soffit dead load, falsework dead load, assumed horizontal load, and the vertical component of the cable unit load. All vertical loads act on the falsework in its deflected position.
- Live load, total dead load of entire superstructure cross section, falsework dead load, and the assumed horizontal load. All vertical loads act on the falsework in its deflected position.

The procedure for box girders structures is as follows:

- Calculate the post loads and the bending moment in the lower cap and upper cap for both loading conditions. Except for symmetrical cable configurations, it will be necessary to determine vertical load eccentricity and post reactions in both transverse directions to find the maximum loads and stresses in the individual posts.
- Investigate posts for both loading conditions:

- Determine the allowable axial compressive stress, $\mathbf{F_a}$, for each post.
- Calculate the axial compressive stress at each post:

$$\mathbf{f_a = \frac{P}{A}} \tag{5-5.09E(1)-1}$$

where $\mathbf{f_a}$ = Axial compressive stress in the post (psi)

P = Post load (lb)

A = cross sectional area of post (in^2)

- Evaluate the post:

$$\mathbf{\frac{f_a}{F_a} \leq 1.0} \tag{5-5.09E(1)-2}$$

where $\mathbf{f_a}$ = Axial compressive stress (psi)

$\mathbf{F_a}$ = Allowable axial compressive stress (psi)

For many cable braced bents, stresses in the lower cap and upper cap may be determined by analysis in the usual manner; which is by using the Case II load combination. This procedure is usually satisfactory because the Case I load combination rarely governs cap beam design. However, if the cables are attached near the end of a cap cantilever supporting a lightly loaded exterior beam, the Case I load combination, which includes the vertical component of the cable design load, may produce the maximum cap bending moment.

5-5.09E(2) Final Steps for Other Bridge Types

The procedure described in the preceding section for box girder bridges is generally applicable to slab and T-beam structures as well, except that for these structure types it is unnecessary to investigate the system for two load cases.

For the calculations above, the design load combination is:

- Design live load, plus total design dead load, plus total horizontal design load, plus the vertical component of the cable unit design load. All vertical loads act on the falsework in its deflected position.

5-5.10 Cable Load Tests

In the absence of sufficient technical data to identify the cable and establish its safe working strength, the contractor may elect to perform one or more load tests. Judgment will be required as to the total number of tests needed.

For example, if the cable type can be identified and if it is in uniformly good condition, a single test may be sufficient for all cable of the same type. However, if the cable cannot be identified, or if it is old and obviously worn, it may be necessary to test each reel or drum furnished.

If a load test is needed to determine the physical properties of the cable, the test must be performed in a qualified testing lab. Field test results are not acceptable because determining cable properties, such as the modulus of elasticity, the elastic stretch, and the net metallic area of the cable requires precise measurements obtainable only with specialized testing equipment.

See Section 7-2, *Load Tests*, for additional information pertaining to all types of load testing.

5-5.11 Splicing Cable

Splicing is prohibited in any cable used as bracing because of the uncertainties associated with cable splicing.

5-5.12 Cable Connection to Shackles

Cables may be connected to shackles with or without thimbles. Thimbles are used to protect the cable from deformation during loading and also increase the efficiency (or strength) of the cable connection.

<u>5-5.12A Cable Connection to Shackles with Thimbles</u>

See manufacturer's recommendations on the reduction in efficiency using thimbles.

<u>5-5.12B Cable Connection to Shackles without Thimbles</u>

When a cable is looped around a shackle pin the efficiency (strength) of the cable is reduced. Refer to Figure 5-20, *Cable Strength Efficiency When Bent Over Pins*, the efficiency (or strength) of the cable is directly related to the ratio of the pin diameter, **D**, and the nominal cable diameter, **d**, by:

$$\mathbf{E = 100 - \frac{50}{\sqrt{R}},\ R \leq 6} \qquad \text{(5-5.12B-1)}$$

$$\mathbf{E = 100 - \frac{76}{R^{0.73}},\ \ R > 6} \tag{5-5.12B-2}$$

$$\mathbf{R = \frac{D}{d}} \tag{5-5.12B-3}$$

where **E** = Cable efficiency (%)

R = Pin to cable ratio

D = Pin diameter (in)

d = Cable diameter (in)

If the pin diameter is the same as the cable diameter, then the ratio, **R**, is one (D/d = 1). Hence, the efficiency (or strength) in the cable loop is 50%. When a cable is looped around a shackle pin, the cable loop is two part around the pin, therefore the load on the cable in the loop is 1/2 of the applied load on the single cable, hence the net efficiency (or strength) of the cable loop is 100%.

Therefore, it is acceptable to connect cables to shackles without thimbles provided that the shackle pin diameter is the same or larger than the cable diameter. The cable diameter must not be larger than 7/8-inch diameter.

The wire rope connection efficiency in Table 5-2, *Wire Rope Connections*, do still apply, hence a cable looped around a shackle with the same diameter and connected with clips has an efficiency of 80%.

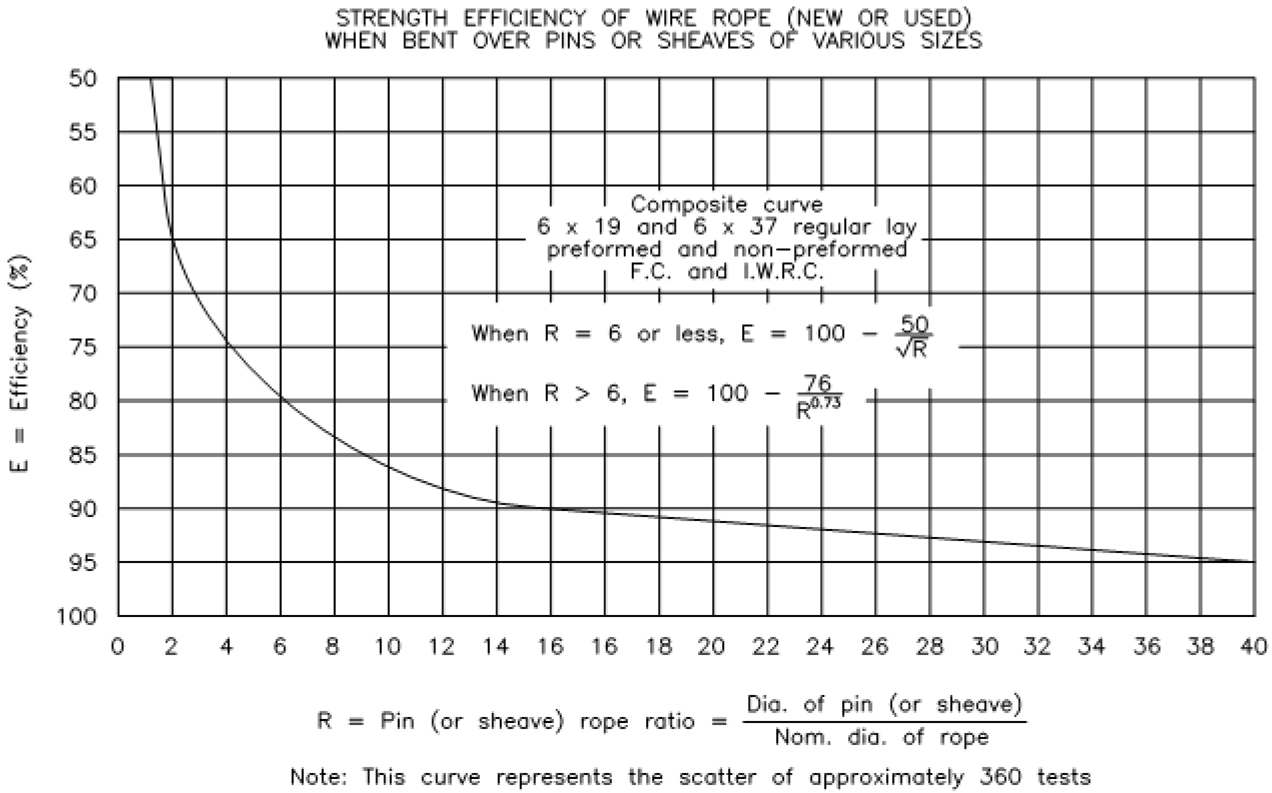


Figure 5-20. Cable Strength Efficiency When Bent Over Pins.

5-5.13 Cable Anchor Systems

In most cases, cables will be secured by fastening the end to a concrete anchor block, although 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.

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

5-5.13A Cable Anchored to Concrete Blocks

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 in Table 5-6, *Coefficient of Friction For Concrete Anchor Blocks.*

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 during the construction period.
- If the blocks are submerged, account for buoyancy effects.

Table 5-6. Coefficient of Friction for Concrete Anchor Blocks.

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

5-6 Short Poured-In-Place Concrete Piles

5-6.01 Introduction

CIDH cable anchors should be evaluated in accordance with the procedure below. Example calculations can be found in Appendix D *Example Problems,* Examples 29 and 30m *Short Poured-In-Place Concrete Piles*.

When CIDH anchors are used, the shop drawings should show:

- Pile diameter and length.
- Cementitious material content for concrete design.
- Reinforcing steel details.
- Cable anchor device.
- Soil pressure and properties.

Since the load resisting capacity of a CIDH anchor is dependent on the characteristics of the soil into which the pile has been cast, the contractor's design calculations should be given a cursory review to determine whether the assumed soil pressure and soil properties are consistent with the type of soil at the job site. Also determine whether the design procedure follows recommended practice for piles subject to both uplift and lateral forces. Any inconsistencies should be brought to the contractor's attention immediately, and supplemental details and/or calculations requested.

Design of piles to resist combined uplift and lateral forces is a sophisticated design procedure, which is sometimes approached superficially in the falsework design. However, the requirement for design calculations in the *Standard Specifications*, Section 48-2.01C(2), *Submittals - Shop Drawings,* applies to piles as well as other elements of the falsework system. In the absence of calculations to support the design, the falsework submittal is not complete and the contractor should be so informed.

The following is a brief review of the technical aspects and a procedure which can be used for investigating rigid piles. This is not a comprehensive coverage of the subject, there are soil complexities not covered, and some caution should be used in its application if primary loads of extended duration are to be supported.

The pile must have the structural capacity to resist tensile, shear, and bending stresses. Reinforcing steel should extend the full length of these piles.

The Division of Engineering Services (DES) Geotechnical Services in Sacramento has furnished the SC criteria for the analysis of loadings on poured-in-place concrete piles. The analysis is dependent on proper selection of soil type. It will be important to determine whether the soil into which the pile is constructed is principally cohesive or cohesionless. Analytical results for pile uplift and lateral loading represent ultimate resistance values.

Load resisting capacity is dependent upon the characteristics of the soil into which the pile has been cast. Preliminary assumptions may be made about soil properties at the time of review of the shop drawings. A final determination of the pile's capacity should be made, however, when the pile hole is excavated and the actual soil can be inspected. The type of soil in the upper third of the hole, its degree of compaction and whether ground water is (or may be), encountered are of primary importance.

Pile loadings are considered in three separate categories:

- Pile uplift
- Lateral loads
- Resistance to combined uplift and lateral loads

Accompanying sample problems are provided in Appendix D *Example Problems,* Examples 29 and 30, *Short Poured-In-Place Concrete Piles*.

5-6.02 Pile Uplift

Pile uplift, acting either vertically or at an angle, is resisted by soil-pile friction (shearing resistance) and the physical weight of the pile.

The shearing resistance of the soil-pile interface is computed differently for cohesive soils than it is for cohesionless soils. The internal angle of friction of the soil is not utilized for poured-in-place piles because in hard ground (high friction angle) the drilling operation loosens the adjacent soil, and in loose ground (low friction angle) the drilling operation tends to compact the adjacent soil particles.

Ultimate pile resistance to uplift is determined by adding the weight of the pile to the quantity of the appropriate unit shearing resistance value multiplied by the surface area of the pile. No additional provisions are made for irregularities along the pile-soil interface.

$$\mathbf{R = \pi dzS + W} \tag{5-6.02-1}$$

where **R** = Resistance to pile uplift (lb)

d = Pile diameter (ft)

z = Depth below ground surface (ft)

S = Unit shearing resistance on the soil-pile interface (psf)

W = Pile weight (lb)

Generally, working load values are to be limited to no more than 1/2 the ultimate load values, which provides a minimum **FS = 2**.

5-6.02A Pile Uplift in Cohesionless Soil

For cohesionless soil, the soil-pile friction (shearing resistance) may be computed using the following equation:

$$\mathbf{S = \beta\sigma_2 \le 4,000\ psf} \tag{5-6.02A-1}$$

$$\mathbf{\beta = 1.5 - 0.315\, z^{1/2}\ but\ 0.25 \le \beta \le 1.2} \tag{5-6.02A-2}$$

where **S** = Soil-pile friction, shearing resistance (psf)

β = Reduction factor for cohesionless soils

$\boldsymbol{\sigma_2}$ = Effective overburden soil weight (psf). Below the water table the weight of water is subtracted from the soil unit weight so that only the submerged soil weight is used

5-6.02B Pile Uplift in Cohesive Soil

The soil-pile friction equations for cohesive soils differ substantially for pile penetrations of less than 5 feet versus piles over 5 feet in depth. The equations also depend on whether the pile is greater or less than 18 inches in diameter:

$$\mathbf{R_s = \pi dz}S \qquad (5\text{-}6.02B\text{-}1)$$

$$\mathbf{S = a_z C \leq 5,500\ psf} \qquad (5\text{-}6.02B\text{-}2)$$

where $\mathbf{R_s}$ = Shearing resistance (lb)

d = Diameter of the pile (ft)

z = Depth below ground surface (ft)

S = Unit shearing resistance (psf)

$\mathbf{a_z}$ = An empirical unitless reduction factor derived from load testing which accounts for clay shrinkage and lateral pile loadings. This variable depends on the depth of pile penetration, having one value for a depth up to 5 feet, and another for penetration over 5 feet

C = Soil cohesion (undrained shear strength) (psf)

Reduction factor, $\mathbf{a_z}$, for pile diameters d > 18":

- The reduction factor, $\mathbf{a_z}$, for the first 5 feet of penetration is 0. The reduction factor remains constant at a_z = 0.55 for all depths greater than 5 feet. This may be expressed in equation form as:
  1. For short piles, 5' or less embedment:

$$\mathbf{a_{z(0-5)} = 0 \quad for\ 0 \leq z \leq 5\ feet} \qquad (5\text{-}6.02B\text{-}3)$$

  2. For pile lengths with more than 5' embedment:

$$\mathbf{a_{z(0-5)} = 0 \quad for\ 0 \leq z \leq 5\ feet} \qquad (5\text{-}6.02B\text{-}4)$$

$$\mathbf{a_{z(>5)} = 0.55 \quad for\ z > 5\ feet} \qquad (5\text{-}6.02B\text{-}5)$$

Reduction factor, $\mathbf{a_z}$, for pile diameters d ≤ 18":

- The reduction for the top 5 feet of pile varies from 0 at z = 0 feet to 0.55 at z = 5 feet, then remains constant at 0.55 for all depths greater than 5 feet. For lengths of pile between 0 and 5 feet, prorate the reduction factor. This concept may be expressed in equation form as:

1. For short piles, 5 feet or less embedment:

$$\mathbf{a_{z(0-5)}} = \left(\frac{\mathbf{0+0.55}}{\mathbf{2}}\right)\frac{\mathbf{z}}{\mathbf{5}} = (\mathbf{0.275})\frac{\mathbf{z}}{\mathbf{5}} = \mathbf{0.055z} \qquad \text{(5-6.02B-6)}$$

2. For pile lengths with more than 5 feet embedment

$$\mathbf{a_{z(0-5)}} = \left(\frac{\mathbf{0+0.55}}{\mathbf{2}}\right)\frac{\mathbf{z}}{\mathbf{5}} = (\mathbf{0.275})\frac{\mathbf{z}}{\mathbf{5}} = \mathbf{0.055z} \qquad \text{(5-6.02B-7)}$$

$$\mathbf{a_{z(>5)}} = \mathbf{0.55} \qquad \text{(5-6.02B-8)}$$

5-6.03 Lateral Loads

Tests have shown that soil resistance to lateral pile loading is greater than that predicted by Rankine equations. For clays the ultimate passive resistance can be as large as 9 times the shear strength, **C**, and for cohesionless soils the ultimate resistance can be 3 times as large as computed Rankine values. The soil resistance acting on isolated piles to a lateral force applied at or near the ground surface may be somewhat depicted as shown in Figure 5-21, *Pile Soil Resistance*.

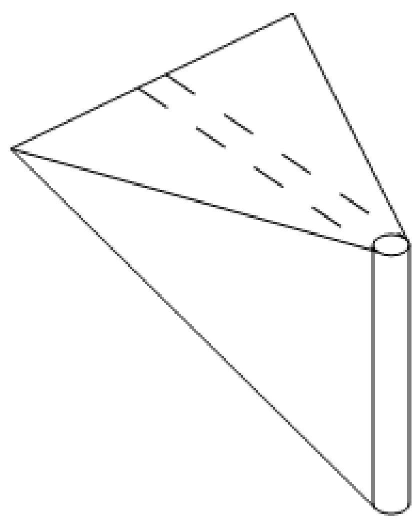


Figure 5-21. Pile Soil Resistance.

Convenient concepts and equations have been developed by Broms for cohesive and cohesionless soils. If a few important soil properties are known, or can be determined, it is possible to compute soil resisting values and pile moments resulting from the application of lateral pile forces at or near the ground surface.

Ultimate pile resistance to lateral loading may be determined by failure of the soil along the total pile length in the case of short piles, or by the yield moment of the pile itself for longer piles. Short unrestrained piles are those piles having a length to diameter ratio of **$L/d \leq 20$** providing the yield moment, $\mathbf{M_y}$, of the pile will be greater than the maximum resisting moment, $\mathbf{M_{ULT}}$, furnished by the soil.

Embedment of piles should be a minimum of 4 times the pile diameter to achieve sufficient soil resisting capacity. The point of rotation of rigid short piles may be assumed to occur between 0.70 to 0.75 times the embedded length; where the larger value coincides with the largest lateral loadings. Soil resisting values are determined by using the lateral resisting value of up to 3 times the passive coefficient, $\mathbf{K_p}$, for cohesionless soil and as much as 9 times the undrained shear strength, $\mathbf{C_u}$, of cohesive soils.

Piles may be considered to act individually provided the pile spacing exceeds 4 pile diameters (4d). When piles are spaced closer than 2 pile diameters (2d), the piles and the soil within the pile group may be considered to act as a single unit.

As piles under load deflect they place the forward soil in a passive condition. When a pile is in clay, a void will be left behind the pulled pile until the clay crumbles or swells. When a pile is in granular material, the soil will soon fill the void behind the pulled pile. When a pile is unloaded, it will generally not return to its original position; some of the pulled deflection will remain. It can readily be seen that unloading and reloading a pile greatly decreases the soils moment capacity for that pile.

Generally, working load values are to be limited to no more than 1/2 the ultimate load values, which provides a minimum **FS = 2** assuming a one time loading of the soil around the pile.

For each subsequent time a pile is to be loaded in the same direction, an additional safety factor of 0.25 is to be added to the previous value as defined by the following:

$$\mathbf{FS = 2 + 0.25(x - 1)} \qquad (5\text{-}6.03\text{-}1)$$

where **FS** = Factor of safety

x = Number of uses in the same direction for the same horizontal component

5-6.03A Lateral Loading in Cohesionless Soil

Considerations used for piles in cohesionless soil include increasing the Rankine passive resistance by a factor of 3, ignoring active pressures on the back side of the pile, and assuming that soil along the total length of buried pile provides resistance at the moment of loading.

Figure 5-22, *Soil Reaction and Bending Moment (Cohesionless Soil),* depicts soil reaction and pile bending moment diagrams for short and long isolated piles in cohesionless soil. The passive resistance at the toe of the short piles is replaced by a concentrated load acting at the pile tip to simplify the moment equation. A plastic hinge is assumed for long piles and the maximum bending moment will be limited to the yield moment, $\mathbf{M_y}$, of the pile.

The maximum moment for short piles occurs at the location of zero shear. For granular soils this plane of zero shear is located at a pile depth of $\mathbf{e + f_g}$ below the plane of application of the lateral load. The distance $\mathbf{f_g}$ equals the length from the ground surface to the plane of zero-shear.

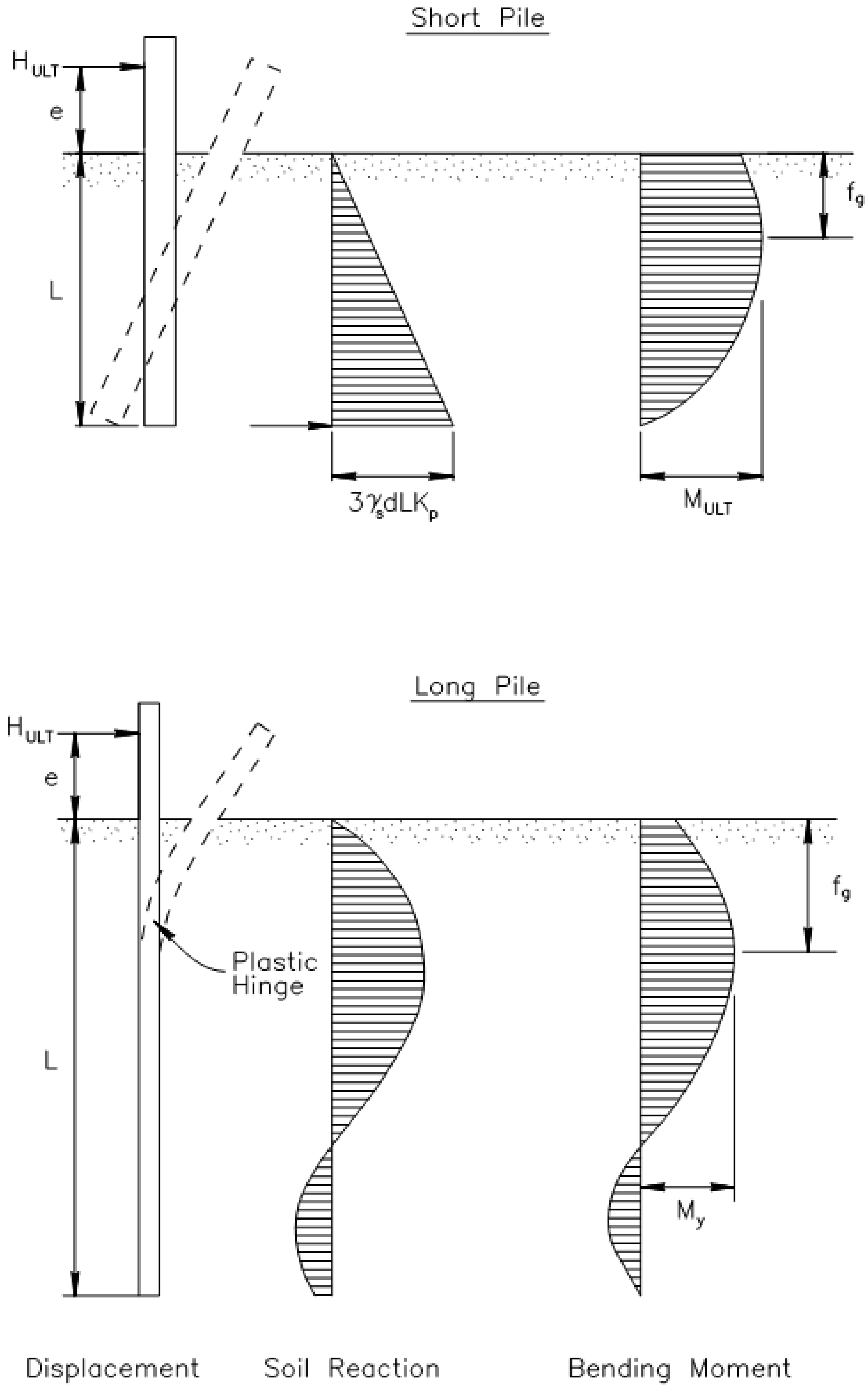


Figure 5-22. Soil Reaction and Bending Moment (Cohesionless Soil).

Equating lateral forces gives:

$$\left(\mathbf{f_g}\right)^2 = \frac{\mathbf{H_{ULT}}}{\mathbf{1.5\gamma_s dK_p}} \qquad \text{(5-6.03A-1)}$$

where $\mathbf{f_g}$ = Length from the ground surface to the plane of zero-shear (ft)

$\mathbf{H_{ULT}}$ = Ultimate lateral load (lb)

γ_s = Soil density (pcf)

d = Pile diameter (ft)

$\mathbf{K_p}$ = Passive coefficient for cohesionless soil

Based on failure of the soil, the maximum moment occurs at a depth of $\mathbf{e + f_g}$:

$$\mathbf{M_{ULT}} = \mathbf{H_{ULT}}\left(\mathbf{e} + \frac{\mathbf{2f_g}}{\mathbf{3}}\right) \qquad \text{(5-6.03A-2)}$$

where $\mathbf{M_{ULT}}$ = Ultimate moment (ft-lb)

$\mathbf{H_{ULT}}$ = Ultimate lateral load (lb)

e = Length from ground surface to ultimate lateral load (ft)

$\mathbf{f_g}$ = Length from the ground surface to the plane of zero-shear (ft)

If the ultimate moment, $\mathbf{M_{ULT}}$, is calculated to be greater than the pile limiting yield moment, $\mathbf{M_y}$, a long pile is indicated and therefore $\mathbf{H_{ULT}}$ must be limited by using $\mathbf{M_{ULT} = M_y}$.

Figure 5-23, *Pile Ultimate Lateral Resistance (Short Piles)* contains curves developed by Broms which relate the pile embedment length ratio, **L/d**, to the ultimate lateral soil resistance for various **e/d** ratios. From this figure $\mathbf{H_{ULT}}$ can be determined for short piles.

Figure 5-24, *Pile Ultimate Lateral Resistance (Long Piles),* may be used for long piles. Broms' curves for values of **e/d** relate the soil ultimate lateral resistance to the yield moment of the pile. This figure is used when the pile embedment length ratio **L/d > 20** and when the yield moment of the pile is less than the ultimate lateral soil resistance.

The maximum safe single use working load for free headed piles in cohesionless soils may be taken as 1/2 of the ultimate load values.

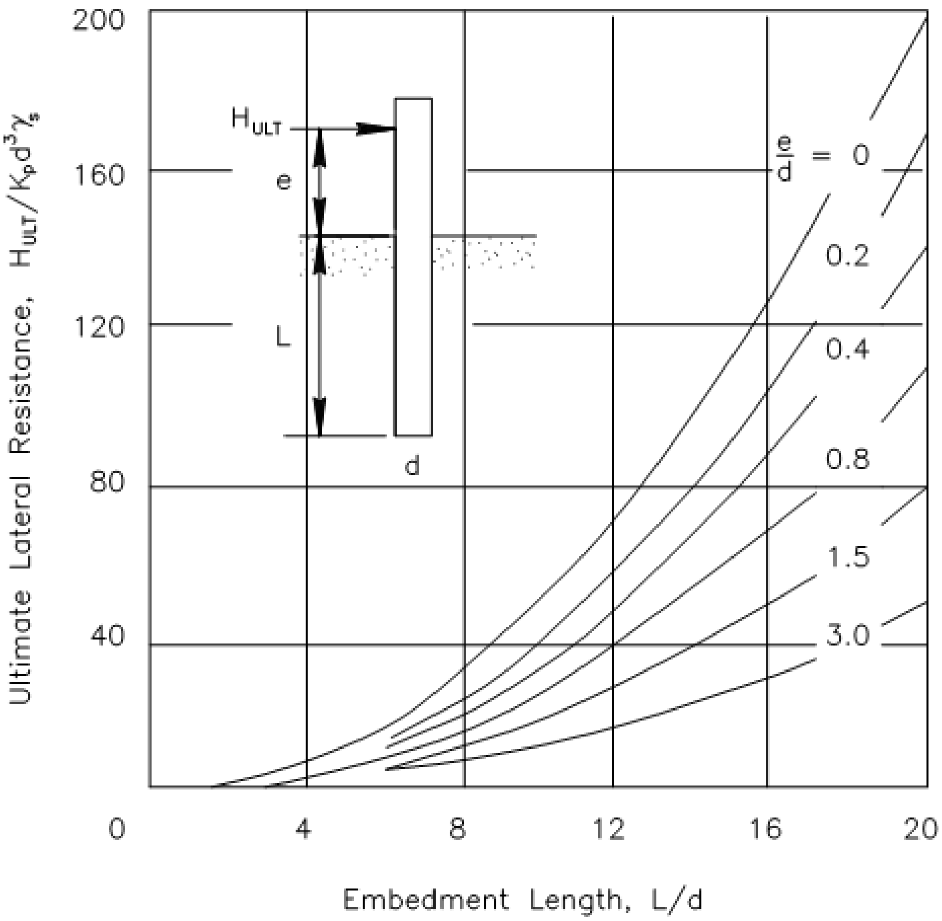


Figure 5-23. Pile Ultimate Lateral Resistance (Short Piles)

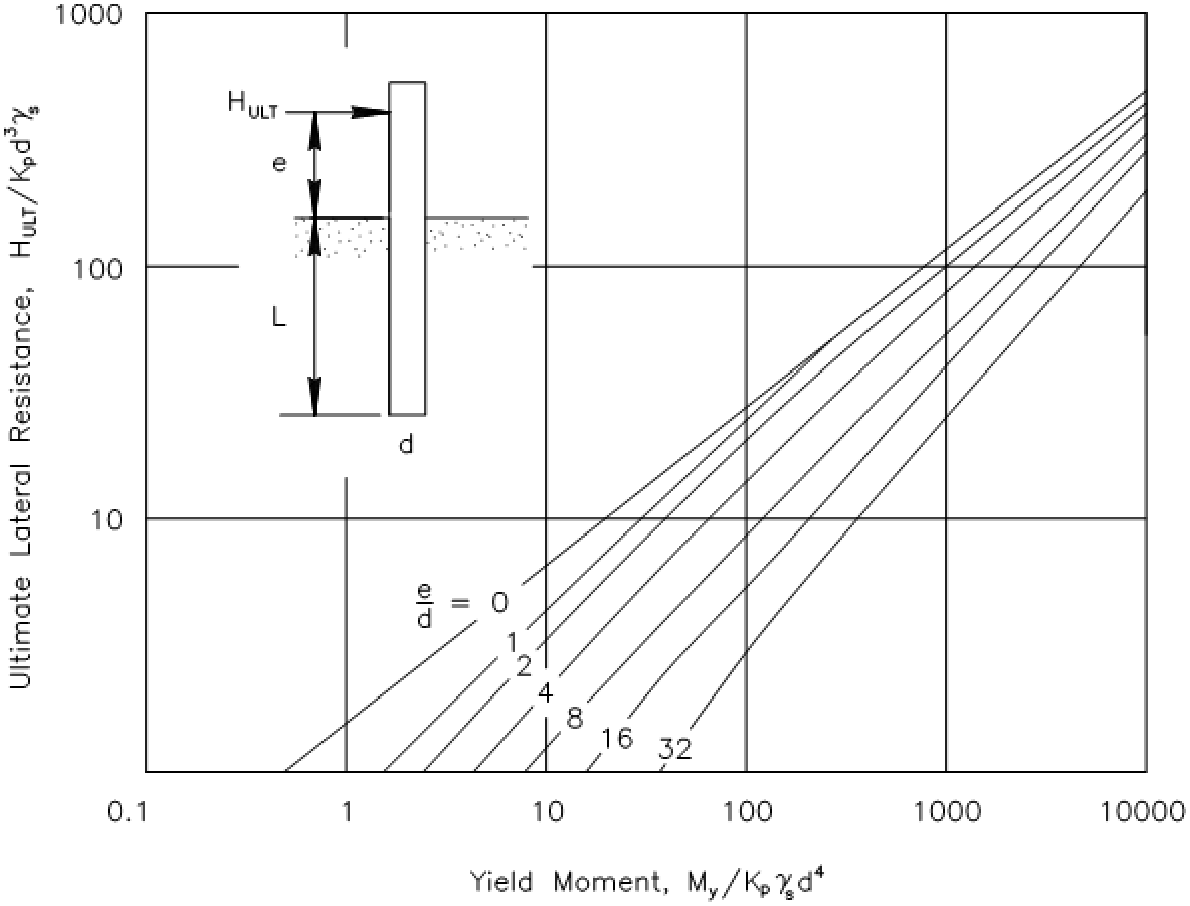


Figure 5-24. Pile Ultimate Lateral Resistance (Long Piles)

5-6.03B Lateral Loading in Cohesive Soil

The ultimate soil resistance for piles in cohesive soils increases to some maximum value at approximately 3 pile diameters below the ground surface then remains fairly constant at greater depth. Literature suggests using a soil distribution of zero between ground surface and a depth of 1.5 times the pile diameter (**1.5d**) and then using a value of 9 times the undrained shear strength (**$9C_u$**) for the remainder of the pile depth.

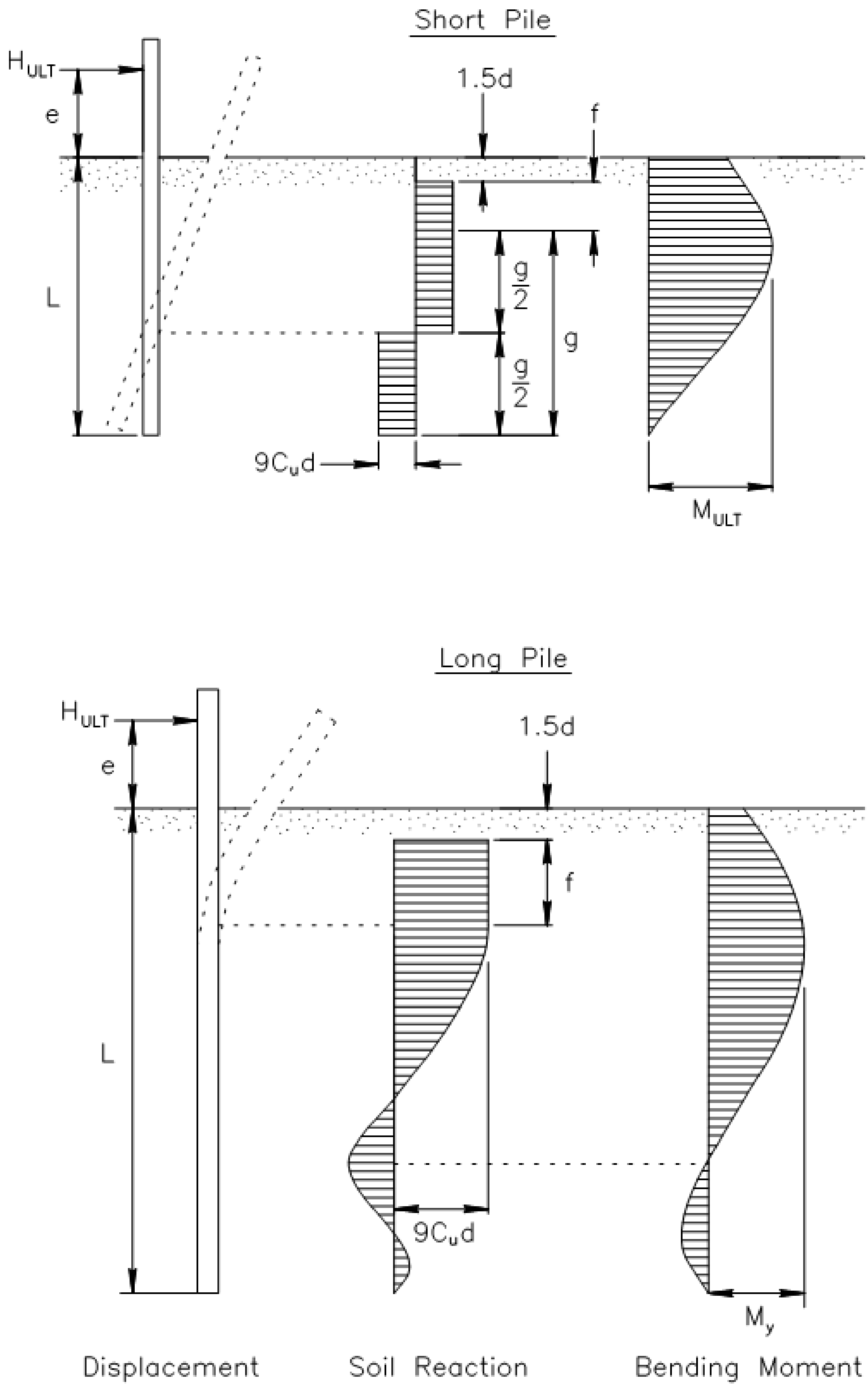


Figure 5-25. Soil Reaction and Bending Moment (Cohesive Soil).

Figure 5-25, *Soil Reaction and Bending Moment (Cohesive Soil),* depicts soil reaction and bending moment diagrams for short and for long piles in cohesive soils. Short piles have a limiting embedment length ratio of **L/d = 20**. Piles having ratios **L/d > 20** are considered to be long piles. For long piles a plastic hinge is assumed in the vicinity of the maximum moment. The yield moment, $\mathbf{M_y}$, of long piles will generally limit the soil resisting maximum moment, $\mathbf{M_{ULT}}$, so that $\mathbf{M_{ULT} = M_y}$ should be used.

The maximum moment for short piles occurs at the location of zero shear. For cohesive soils the plane of zero shear is located at a pile depth of $\mathbf{e + 1.5d + f_c}$ below the plane of application of the horizontal force. The distance $\mathbf{f_c}$ develops from equating horizontal forces:

$$\mathbf{f_c = \frac{H_{ULT}}{9C_u d}} \qquad \text{(5-6.03B-1)}$$

where $\mathbf{f_c}$ = Length from 1.5d below ground surface to point of zero shear (ft)

$\mathbf{H_{ULT}}$ = Ultimate lateral load (lb)

$\mathbf{C_u}$ = Undrained shear strength (psf)

d = Pile diameter (ft)

Based on failure of the soil, the maximum moment occurs at a depth of $\mathbf{e + 1.5d + f_c}$ and the maximum moment is:

$$\mathbf{M_{ULT} = H_{ULT}(e + 1.5d + 0.5f_c)} \qquad \text{(5-6.03B-2)}$$

where $\mathbf{M_{ULT}}$ = Ultimate moment (ft-lb)

$\mathbf{H_{ULT}}$ = Ultimate lateral load (lb)

e = Length from ground surface to ultimate lateral load (ft)

d = Pile diameter (ft)

$\mathbf{f_c}$ = Length from (1.5d) below ground surface to point of zero shear (ft)

If the ultimate moment, $\mathbf{M_{ULT}}$,is calculated to be greater than the yield moment, $\mathbf{M_y}$, of the pile, a long pile is indicated and $\mathbf{H_{ULT}}$ must be limited by using $\mathbf{M_{ULT} = M_y}$.

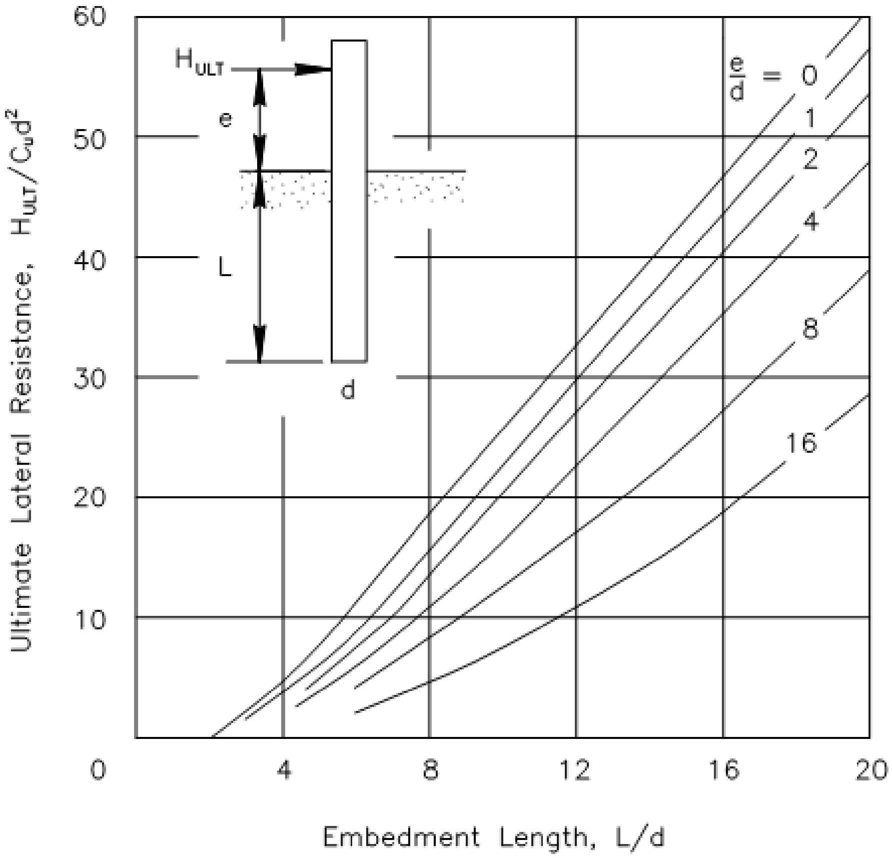


Figure 5-26. Ultimate Lateral Resistance and e/d Ratio (Short Piles).

Figure 5-26 *Ultimate Lateral Resistance and e/d Ratio (Short Piles)* contains curves developed by Broms for short piles which relate the pile embedment depth ratio, **L/d**, to the ultimate lateral soil resistance for various **e/d** ratios.

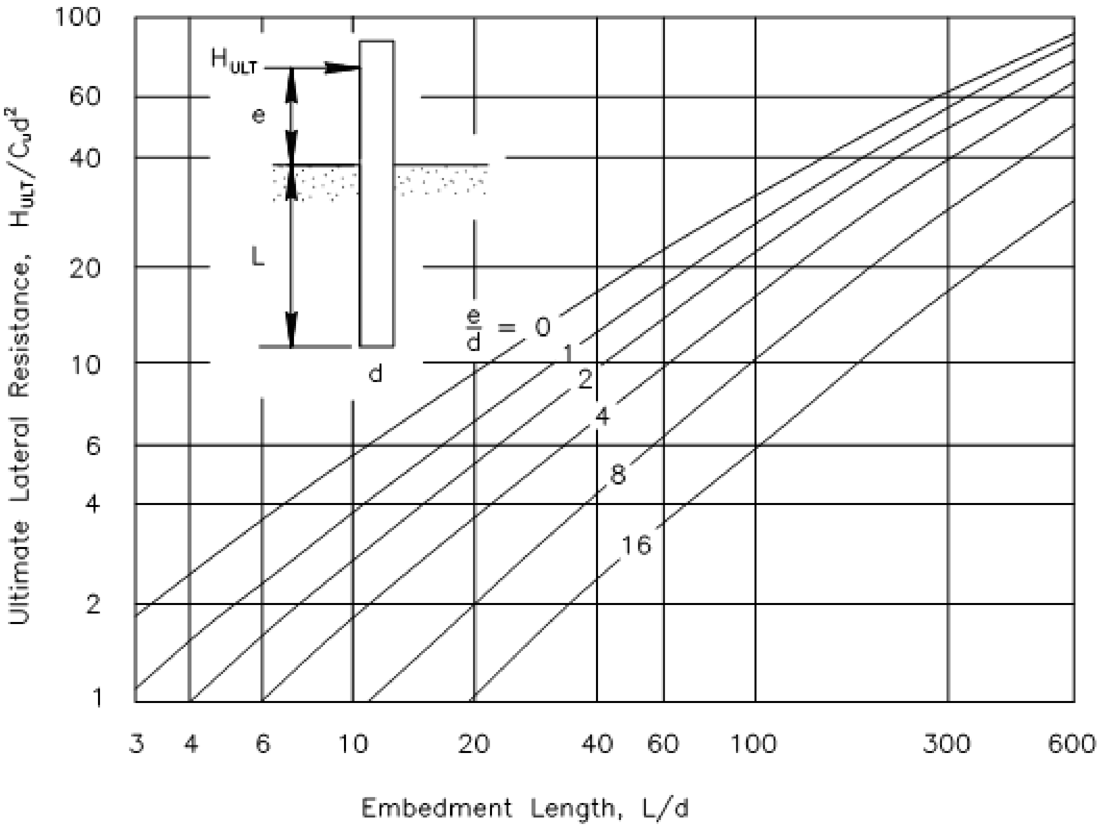


Figure 5-27. Ultimate Lateral Resistance and e/d Ratio (Long Piles).

Figure 5-27 *Ultimate Lateral Resistance and e/d Ratio (Long Piles)* may be used for long piles. Curves developed by Broms for **e/d** values relate the soils ultimate lateral resistance to the yield moment of the pile. This figure is used when the pile embedment length ratio **L/d > 20** and when the yield moment of the pile is less than the moment due to the ultimate lateral soil resistance.

The safe single use working load for free headed piles in cohesive soil may be taken as 1/2 of the ultimate load value.

5-6.04 Concrete Stresses

Concrete stresses in the pile may be computed by rigorous analysis; or may be approximated by assuming an average compressive condition over half of the pile width. The maximum compressive stress is located on one face of the pile. It should be assumed that the concrete will not take tensile forces on the other half of the pile. Tensile forces will be resisted by reinforcing steel.

For simplified compressive stress analysis use:

$$\mathbf{f_c} = \frac{\mathbf{Md}}{\mathbf{2I_g}} - \frac{\mathbf{V'}}{\mathbf{A_g}} \leq \frac{\mathbf{f'_c}}{\mathbf{2}} \qquad \text{(5-6.04-1)}$$

where $\mathbf{f_c}$ = Concrete compressive stress (psi)

M = Maximum moment in pile (in-lb)

d = Pile diameter (in)

$\mathbf{I_g}$ = Moment of Inertia on the gross pile section (in^4)

$\mathbf{A_g}$ = Gross cross sectional area of the pile (in^2)

V'= Tensile force (vertical force component) less pile weight above plane of zero shear (lb). Distance from pile top to the plane of zero shear is defined as MULT/HULT

$\mathbf{f'_c}$ **=** Concrete compressive strength

The computed maximum compressive stress, $\mathbf{f_c}$, shall not be greater than 1/2 of the concrete compressive strength, $\mathbf{f'_c}$, anticipated at the time the pile is to be loaded.

The allowable shear in the pile, $\mathbf{V_u}$. normal to the pile should not exceed 2 times the square root of concrete compressive strength $\left(2\sqrt{f'_c}\right)$.

5-6.05 Bar Reinforcing Stresses

Bar reinforcing steel stresses may be analyzed by rigorous methods; or may be approximated by making several assumptions.

Ignore concrete stress and assume the pile moment is to be resisted by the reinforcing steel. For symmetrical reinforcing it can be assumed that the reinforcing takes compression as well as tension. A simplified equation may be used to determine the tensile reinforcing steel stress.

$$\mathbf{f_s} = \frac{\mathbf{M d_s}}{\mathbf{2 I_{bars}}} + \frac{\mathbf{V'}}{\mathbf{\Sigma A_s}} \qquad \text{(5-6.05-1)}$$

$$\mathbf{I_{bars}} = \Sigma\left[\mathbf{I_0} + \mathbf{A_s}\left(\frac{\mathbf{d_s}}{2}\right)^2\right] \approx \Sigma \mathbf{A_s}\left(\frac{\mathbf{d_s}}{2}\right)^2 \qquad \text{(5-6.05-2)}$$

where: $\mathbf{f_s}$ = Tensile stress in reinforcing steel (psi)

M = Maximum moment in pile (in-lb)

$\mathbf{d_s}$ = Distance between center of gravity of bars either side of the pile neutral axis (in)

V' = Tensile force (vertical force component) less weight of pile above plane of zero shear (lb), which is located a distance of $\mathbf{M_{ULT}/H_{ULT}}$ below the pile top.

$\mathbf{A_s}$ = Area of reinforcing steel on either side of the neutral axis (in^2)

For 2 reinforcing bars, one either side of the pile center line symmetrically placed, the simplified equations is:

$$\mathbf{f_s = \frac{M\frac{d_s}{2}}{2A_s\left(\frac{d_s}{2}\right)^2} + \frac{V'}{\Sigma A_s} = \frac{M}{A_s d_s} + \frac{V'}{\Sigma A_s}} \qquad \text{(5-6.05-3)}$$

The allowable stress in the reinforcing steel should be limited to:

$$\mathbf{F_s \leq 0.70F_y} \qquad \text{(5-6.05-4)}$$

5-6.06 Resistance to Combined Uplift and Horizontal Load

Pile load tests have confirmed that the uplift resistance of piling is increased when the pile is also subjected to a lateral loading. Therefore it is believed acceptable to simply limit combined loadings so as not to exceed the permissible $\mathbf{H_{ULT}}$ and $\mathbf{V_{ULT}}$ loadings (safety factors considered).

Design load is limited to the smaller of either $\mathbf{V/\sin\theta}$ or $\mathbf{H/\cos\theta}$. See Figure 5-28, *Load Components for Plumb Piles*.

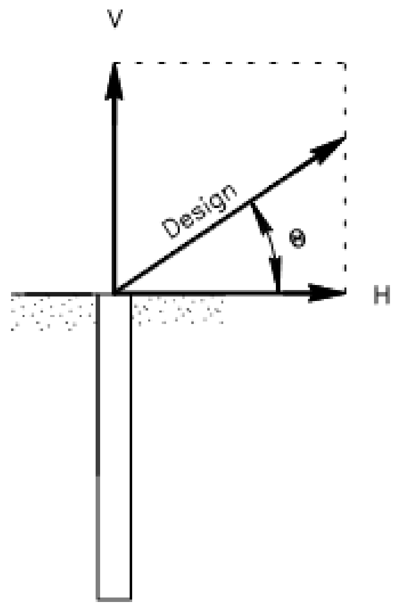


Figure 5-28. Load Components for Plumb Piles.

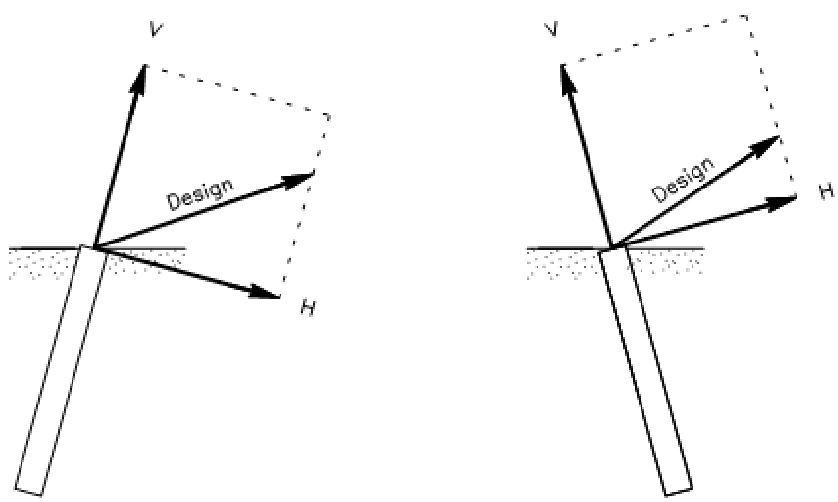


Figure 5-29. Load Components for Battered Piles.

Tests have also demonstrated that when the top of the pile is battered toward the load its lateral capacity is substantially greater than when battered away from the load. Force components **H** and **V** for battered piles are derived in the same manner as for plumb piles. The design load is then limited to the lesser of the **H** or **V** load resolved to the slope at which the design load will be acting. Piles battered toward and away from the design loading are depicted in Figure 5-29, *Load Components for Battered Piles*.

5-7 Combining Stresses

5-7.01 Introduction

As noted elsewhere in this manual, stresses produced by the simultaneous application of horizontal and vertical forces need to be combined in those situations where bending must be considered to prevent overstressing of an axially-loaded member of the falsework system. Examples of such situations include, but are not limited to, bents braced by cables, moment resisting connections, moment resisting piles, pile bents over water where the bracing extends only to the water surface, and multi-tiered frame bents where the bracing system, although adequate to resist the collapsing force, does not fully support the vertical members in the bent and/or cannot prevent side sway.

5-7.02 Design Criteria

The ability of a falsework member to resist the combined effect of bending and axial compression is evaluated by the combined stress expression. The combined stress expression, or interaction formula as it is sometimes called, establishes a limiting relationship between bending and compressive stresses, such that the sum of the actual/allowable ratios of the two stresses may not exceed 1.0. In formula form the combined stress expression is:

$$\frac{\mathbf{f_b}}{\mathbf{F_b}} + \frac{\mathbf{f_c}}{\mathbf{F_c}} \leq \mathbf{1.0} \qquad (5\text{-}7.02\text{-}1)$$

where $\mathbf{f_b}$ = Calculated bending stress

$\mathbf{f_c}$ = Calculated compressive stresses

$\mathbf{F_b}$ = Allowable bending stress

$\mathbf{F_c}$ = Allowable axial compressive stress

The combined stress expression may be used to determine the adequacy of members to resist bending and axial compression in all cases except driven timber piles. Timber piles should be evaluated in accordance with the procedures discussed in Chapter 8, *Foundations*.

5-8 Longer T-Beam Falsework Spans

5-8.01 Introduction

It is acceptable to exceed the specified falsework span length provided the criteria below is satisfied. For additional information about T-Beams see Section 4-6, *T-Beam Bridges*.

5-8.02 Design Criteria

Longer T-Beam falsework spans may only be considered if the deflection due to concrete loading in the longer span is the same as the maximum deflection for the specified falsework span length. To fulfill this requirement, the moment of inertia of the longer span stringer must be greater than that required for a stringer for the specified span length.

The moment of inertia of the longer falsework stringer can be found by equating the deflection of the shorter and longer spans as shown below.

The deflection of the stringer using the specified span length is:

$$\Delta_1 = \frac{5wL_1^4}{384EI_1} \quad (5\text{-}8.02\text{-}1)$$

$$L_1 = 12(14 + 8.5D) \quad (5\text{-}8.02\text{-}2)$$

where Δ_1 = Deflection of falsework stringer using specified span length (in)

w = Uniform load on stringer (lb/in)

L_1 = Specified falsework span length (in)

E = Modulus of elasticity (psi)

I_1 = Moment of inertia of stringer for specified span length (in^4)

D = T-Beam depth measured from top of deck to bottom of girder (ft). For T-beams with varying depth (haunch) use the minimum depth

The deflection of the stringer using the proposed longer falsework span length is:

$$\Delta_2 = \frac{5wL_2^4}{384EI_2} \quad (5\text{-}8.02\text{-}3)$$

where Δ_2 = Deflection of falsework stringer using proposed span length (in)

w = Uniform load on stringer (lb/in)

L_2 = Proposed falsework span length (in)

E = Modulus of elasticity (psi)

I_2 = Moment of inertia of stringer for proposed span length (in^4)

Equating the two deflections and solving for I_2 yields the required moment of inertia for the falsework stringer for the proposed span length:

$$\mathbf{I_2} = \mathbf{I_1}\left(\frac{\mathbf{L_2^4}}{\mathbf{L_1^4}}\right) \qquad (5\text{-}8.02\text{-}4)$$

Chapter 6: Stability

Table of Contents

6-1 Introduction

The term stability, as it is used throughout this manual, means resistance to overturning or collapse of the falsework system or elements of the system under consideration. Resistance to both overturning and collapse is provided by the falsework bracing system, which must be designed to withstand all forces resulting from application of the assumed horizontal load.

The term falsework bracing system as it is used in the specifications includes bracing designed to resist overturning or collapse. Falsework bracing systems are usually comprised of struts, ties, cables, anchor blocks, and similar features used to prevent the overturning or collapse of any falsework component. Regardless of functional elements, the bracing system must be designed to resist all forces generated by the assumed horizontal load.

It is important to recognize the distinction between overturning and collapse, as these terms are used to describe the failure modes when falsework is subjected to horizontal forces. Overturning is used when the bracing provides sufficient rigidity to the system as a whole, or to the element of the system under consideration, so that the system or element acts as a single, rigid unit. In such cases the falsework will fail by overturning, or rotation about its base. If, however, the bracing cannot prevent distortion of the falsework when it is subjected to horizontal forces, the system will collapse internally rather than overturn. The two failure modes are shown schematically in Figure 6-1, *Falsework Failure Modes*.

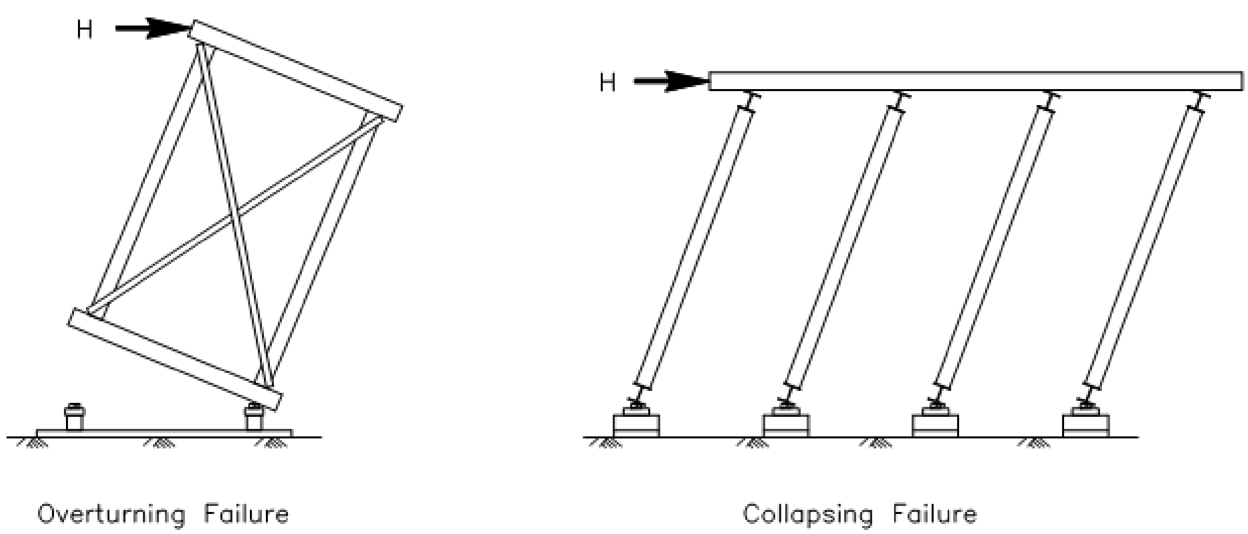


Figure 6-1, Falsework Failure Modes.

The specifications do not require the falsework to carry the assumed horizontal load from its point of application through all members of the system to the ground or other point of support. If the bracing system will resist the overturning and collapsing forces produced by the horizontal design load, then the design complies with the intent of the specifications.

For conventional falsework, experience has shown that the possibility of sliding failure is low. However, sliding failure may occur and engineering judgment should be used when designing and reviewing the falsework. Failure due to sliding is more likely to happen in the unloaded stage of falsework. Sliding must also be considered in each plane between falsework tiers where the posts are discontinuous.

When cables are released for grading or adjustment, pork-chops and come-alongs, or similar systems, must be used to control the release of the cable and to maintain stability of the system. Loosening the clips without control is not acceptable. The system being used to release cables must be shown on the shop drawings.

Stresses in falsework members produced by the application of a horizontal force should be combined with stresses produced by vertical forces to provide stability. This results in additional stresses in an axially loaded member thus decreasing the stability of the system. For additional information see Section 5-7, *Combining Stresses*.

6-2 Inherent Stability

Some falsework systems have inherent stability due to their material properties and geometry used in their construction. For example, timber bents have a degree of inherent resistance to collapse, particularly where short, wide posts are used.

Since the amount of inherent stability developed under a given loading condition is a very intangible factor, therefore, the inherent ability of a falsework frame to resist overturning and collapsing forces must be neglected in all cases where:

$$\mathbf{L} > \mathbf{3b} \qquad (6\text{-}2\text{-}1)$$

where **L** = Height of post

b = Width of post

When post height exceeds the limiting ratio, resistance to overturning and/or collapse must be provided by diagonal bracing, blocking, ties, or other means authorized.

6-3 Diagonal Bracing

6-3.01 Introduction

In conventional falsework systems, the individual posts that make up the falsework bent are stabilized against collapse by wood, steel, and cable diagonal cross bracing. The stiffer diagonal braces of wood and steel are installed across two or more vertical posts and securely nailed, bolted, or welded in place to make a single rigid unit capable of resisting the collapsing forces produced by horizontal loads. The more flexible cable bracing is typically installed diagonally from top cap to bottom cap.

6-3.02 Wood Cross Bracing

Studies of the behavior of braced falsework bents have revealed that the actual load imposed on the compression members, under certain loading conditions, may be as much as 2 times greater than would be indicated by a simplified analysis. This is due to the indeterminate nature of braced falsework bents and due to the material properties of wood. To ensure the compatibility of results obtained by our procedure with results obtained by a rigorous frame analysis:

- The contribution of the compression members, and the compression member connections is limited to 1/2 of their theoretical contribution when calculating the resisting capacity of the bracing system

SC has developed a review procedure which simplifies the analysis, and at the same time minimizes the risk of detrimental overstressing of the compression members. This simplified procedure, called the “resisting-capacity” method of analysis, assumes that the collapsing force produced by the assumed horizontal load will be resisted by the sum of the horizontal components of the allowable load carrying capacities of the diagonals. To ensure stability, the sum of the horizontal components (i.e., the resisting capacity of the diagonal braces) must be numerically equal to or larger than the collapsing force.

When compression members have intermediate fasteners to reduce the unsupported length for design, the fasteners must be capable of resisting a force, applied at right angles to the member, equal to:

- 5% of the theoretical design capacity of the member
- Not less than 250 lbs.

When the bracing members are required to be bolted to the main members, other fastener types may be used as intermediate fasteners in the center of a diagonal brace.

To ensure uniformity, the adequacy of diagonal bracing must be checked by the resisting-capacity method. The procedure depends on the number of vertical tiers of bracing used in the bent, as discussed in the following two sections.

6-3.02A Analysis of Single Tier Framed Bents

A single tier framed bent consists of two or more posts braced with one level of diagonal cross bracing.

For single-tier bracing, the resisting capacity of the diagonal bracing system is calculated as follows, regardless of the type of fastener (nails, bolts, or lag screws) used in the connection:

1. Determine the strength of the connection between brace and post. The strength value will be the same for both tension and compression members. For this calculation, follow the procedure in Section 5-3, *Timber Fasteners*, for the type of fastener used.
2. Determine the strength of the diagonal braces in tension.
3. Compare the strength value of the connection (from step 1) and the strength of the brace in tension (from step 2). The smaller of these two values is the strength of the tension members.
4. Calculate the horizontal component of the strength value found in step 3. The horizontal component is the resisting capacity of the tension members.
5. Determine the strength of the diagonal braces in compression, as limited by the Column Stability Factor, $\mathbf{C_P}$.
6. Compare the strength of the connection (from step 1) and the strength of the braces in compression (from step 5). The smaller of these two values is the theoretical strength of the compression members. 1/2 of the theoretical strength is the allowable strength of the compression members.
7. Calculate the horizontal component of the allowable strength (step 6) to obtain the resisting capacity of the compression members.
8. Add the resisting capacity of all tension members and all compression members to obtain the total resisting capacity of the diagonal bracing system.

Compare the total resisting capacity of the diagonal bracing system, as determined above, and the assumed horizontal load applied to the falsework bent. The resisting capacity of the bracing system must equal or exceed the horizontal force applied in either direction; otherwise the bracing is not adequate.

The resisting capacity method is illustrated in Example 14, *Diagonal Bracing of Single Tier Framed Bent – Nailed Connections,* and Example 15, *Diagonal Bracing of Single Tier Framed Bent – Bolted Connections,* in Appendix D, *Example Problems*.

6-3.02B Analysis of Multi-Tier Framed Bents

A multi-tiered framed bent consists of two or more continuous posts braced with two or more levels of diagonal cross bracing.

When the diagonal bracing system consists of more than a single tier, the collapsing resistance of the frame may be limited by the resisting capacity of any individual tier of bracing within the frame. The resisting capacity of the bracing in each tier must be evaluated independently of the other tiers to ensure that each independently braced element of the bent (i.e. each tier) can withstand the collapsing force applied to that element.

Excess resisting capacity in one tier may not be used to compensate for a deficiency in the resisting capacity of any other tier.

The resisting-capacity method has been verified by analysis of mathematical models of typical and atypical falsework configurations. These analytical studies reveal that a horizontal brace between the tiers in a multi-tiered frame makes only a marginal contribution to the total resisting strength of the frame, and under some loading conditions may actually decrease (although only slightly) the effectiveness of the compression members as compared to similar frames in which no horizontal braces are used. Since horizontal braces appear to be redundant members of the system, their effect on frame capacity may be neglected when checking diagonal bracing by the resisting-capacity method in all cases where the diagonals are capable of resisting compression.

A horizontal brace will be required between tiers in a multi-tiered frame in those cases where the diagonal braces can carry tension forces only. The reason is that braces in tension will create a lateral load on the post, so a strut is needed to stabilize the post to resist buckling.

6-3.02B(1) Falsework Bent with Equal Height Tiers

When the tiers have equal height, the resisting capacity of each tier is the same given the same bracing and connections. Consider the diagonally braced bent shown in Figure 6-2, *Falsework Bent with Equal Height Tiers*. Evaluating the adequacy of the bracing in a bent where the bracing system is the same in each tier, is simplified by symmetry.

The procedure is as follows:

1. Calculate the resisting capacity of the diagonal bracing in either tier (the values are the same for both tiers). Follow the procedure discussed in Section 6-3.02A, *Analysis of Single-Tier Framed Bents*.
2. Compare the total resisting capacity calculated in step 1 and the horizontal force. If the resisting capacity equals or exceeds the horizontal force, the bracing in that tier is adequate, and therefore the bent bracing system is adequate as well.

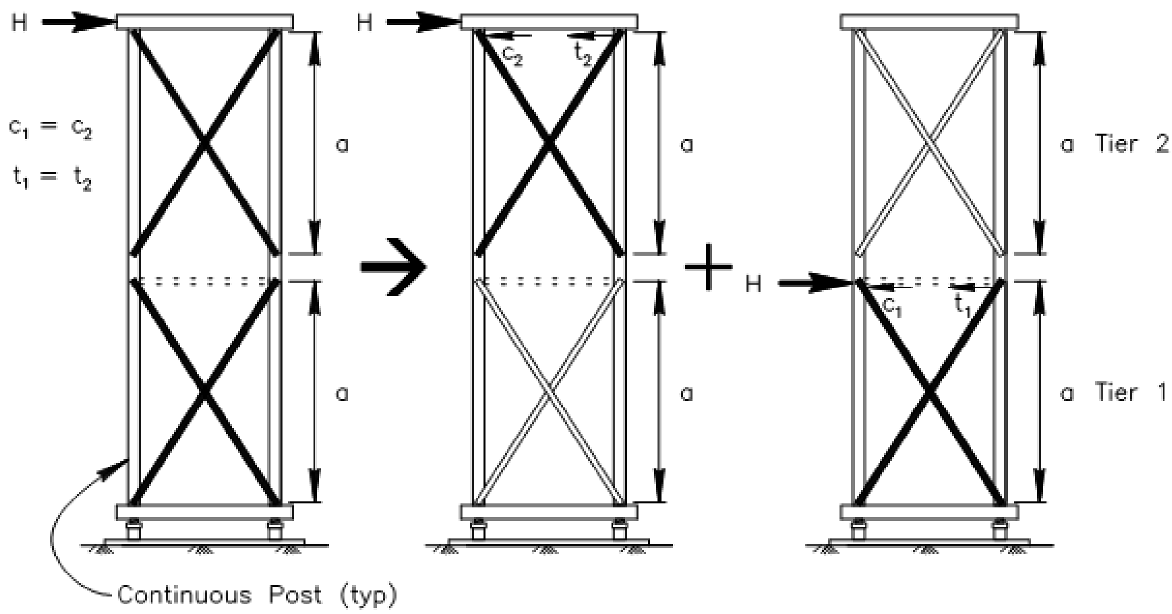


Figure 6-2. Falsework Bent with Equal Height Tiers.

The procedure for evaluating bracing adequacy when the bracing system is the same in each tier, as described herein for a two-tiered bent, will also apply to bents with three, or more, identical tiers of bracing.

6-3.02B(2) Falsework Bent with Unequal Height Tiers

When the tiers are of different heights or are otherwise dissimilar, the resistance provided by the bracing in each tier may not be the same as the resistance provided by bracing in other tiers. Therefore, the resisting capacity of the bracing in each tier must be evaluated independently of the bracing in the other tiers. Consider the diagonally braced bent shown in Figure 6-3, *Falsework Bent with Unequal Height Tiers*.

The procedure is as follows:

1. Calculate the resisting capacity of the bracing in tier 2, following the procedure in Section 6-3.02A, *Analysis of Single-Tier Framed Bents.*

2. Compare resisting capacity and horizontal force. For this comparison, the horizontal force is assumed as acting in a plane through the upper connections in the tier 2 bracing. The resisting capacity of the bracing in tier 2 must equal or exceed the collapsing force.
3. Repeat steps 1 and 2 for tier 1.

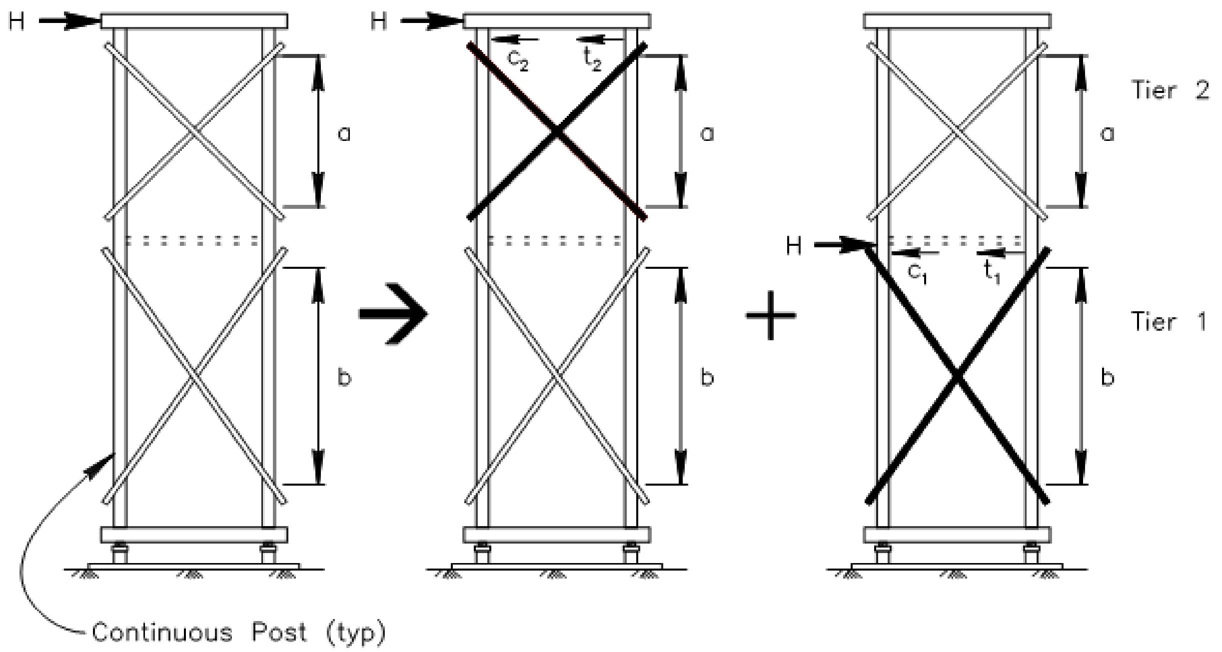


Figure 6-3. Falsework Bent with Unequal Height Tiers.

If the resisting capacity of the diagonal bracing in each tier will withstand the horizontal force applied at that tier, the diagonal bracing system is adequate. If, however, the resisting capacity of either tier is less than the horizontal force, the bracing system is not adequate.

6-3.02B(3) Post Bending

If the tiers of diagonal bracing are closely spaced vertically the effect of bending in the posts between the connections is small and may be neglected when investigating post capacity. If the tiers are separated, however, as shown in Figure 6-4, *Falsework Bent with Separated Tiers,* then bending may be an important factor.

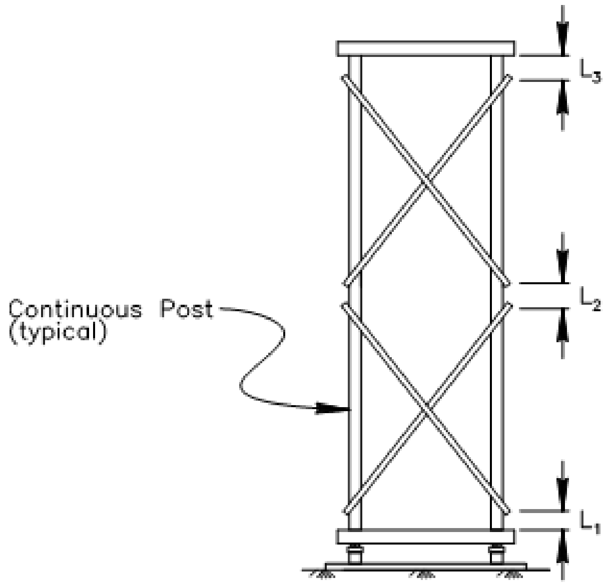


Figure 6-4. Falsework Bent with Separated Tiers.

To ensure uniformity, the effect of bending on post capacity must be investigated if *any* of these conditions are true:

$$\mathbf{L_1 > 4d} \quad \text{(6-3.02B(3)-1)}$$

$$\mathbf{L_2 > 4d} \quad \text{(6-3.02B(3)-2)}$$

$$\mathbf{L_3 > 4d} \quad \text{(6-3.02B(3)-3)}$$

where $\mathbf{L_1}$ = vertical distance below the tiers of bracing

$\mathbf{L_2}$ = vertical distance between the tiers of bracing

$\mathbf{L_3}$ = vertical distance above the tiers of bracing

d = diameter or width of post

When bending in the post is considered, secondary effects due to horizontal deflection (P-Δ effect) must be included. The design must consider the effect of horizontal deflection on member stresses. The analysis for wood posts should follow the procedure for evaluating the adequacy of timber pile bents (see Chapter 8, *Foundations,* except that the posts will be considered as pinned at both the top and bottom.

6-3.03 Steel Bracing

The resisting-capacity method, as discussed in the preceding sections is also applicable when rigid steel bracing (e.g. angle or channel) is used with either steel or timber posts.

Rebar and cable bracing are flexible and are not considered to be rigid bracing. Thus, they do not provide compression resistance in diagonal bracing.

6-3.04 Cable Bracing

For cable bracing see Section 5-5, *Cable Bracing Systems*.

6-4 Longitudinal Stability

6-4.01 Introduction

It is necessary to provide a system of restraint that will prevent the falsework bents from collapsing when the assumed horizontal load is applied in the longitudinal direction. This can be accomplished by diagonal bracing between pairs of adjacent bents, or by transferring the horizontal load from one falsework span to the next falsework span ahead without allowing any horizontal force to reach the bent between the two spans.

6-4.02 Bracing

Consider, for example, the system shown in Figure 6-5, *Braced Falsework System*. Longitudinal forces generated by the assumed horizontal load are carried in either direction across the unbraced bents D and E to the point of longitudinal restraint at bents C and F. The system is stabilized by diagonal bracing between bents B-C and F-G, which are each designed to resist one-half of the total horizontal load acting on the system.

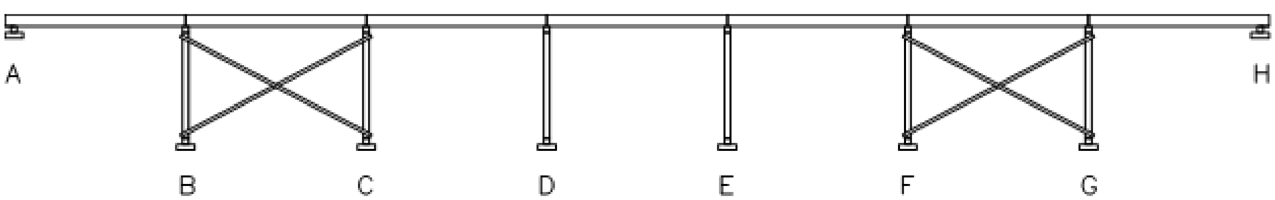


Figure 6-5. Braced Falsework System.

The adequacy of longitudinal bracing used to stabilize adjacent bents will be determined in accordance with the procedure discussed in Section 6-3, *Diagonal Bracing*.

6-4.03 Friction

The method by which the assumed horizontal load is carried across an unbraced bent should be analyzed to verify that horizontal forces can transfer across the unbraced bent under all loading conditions. Many designs will take advantage of frictional resistance between stringer and cap to transfer at least a part of the total longitudinal force acting at the bent. Friction will not be developed until a vertical load is applied. Therefore, in the unloaded condition do not allow more frictional resistance than will be developed by the dead load of the falsework members plus an allowance for the weight of forms and reinforcing steel.

6-4.04 Devices

If frictional resistance alone is not sufficient to withstand the assumed horizontal load, some positive means of restraint must be provided to carry that portion of the total load in excess of the maximum allowable frictional resistance. Positive means of restraint includes blocking, bracing, dowels, clips, cables, and similar mechanical connecting devices capable of transferring horizontal forces in the absence of a vertical load but does not include C-clamps.

Devices used to transfer horizontal forces across an unbraced bent must be spaced far enough apart transversely to prevent eccentric loading on the restraining member. In general, this will require at least two points of mechanical transfer for each independent element of the system. One-point transfer may be acceptable under unusual circumstances, such as a case where the force to be transferred is small when compared to the total horizontal load, or where each independent element is relatively narrow. This is a matter of engineering judgment. In case of doubt, two points of load transfer should be required.

6-5 Overturning

6-5.01 Introduction

If the falsework system, or the element of the system under consideration, is adequately braced to prevent collapse, the system or element may still fail by overturning, or by rotation about its base, when the assumed horizontal load is applied. Overturning failure will occur unless the falsework is inherently stable against overturning by reason of its configuration or is braced to prevent overturning.

In stability analysis, the assumed horizontal load produces a moment that acts to overturn the system or element of the system under consideration. For descriptive purposes, this moment is called the *overturning* moment.

6-5.02 Overturning of Falsework

When calculating overturning moment, the moment arm will be measured from a plane at the top of the falsework member that is set on the ground, and the assumed horizontal load will be applied to the falsework in accordance with the following:

- Actual loads due to equipment, construction sequence, or other causes, will be considered as acting at the point of application on the falsework.
- Wind loads will be considered as acting at the centroid of the wind impact area for each height zone. When wind loads govern the design, however, the assumed horizontal load, used in calculating the overturning moment, is applied in a plane at the top of the falsework post or shoring. See Section 3-3.03, *Wind Loads*.
- When the minimum load governs the design, it is assumed as acting in a plane at the top of the falsework posts or shoring.

6-5.03 Overturning of Elements or Systems of the Falsework

When calculating the overturning moment acting on other elements or systems of the falsework where stability is a factor for consideration, such as a pony bent system, the moment arm will be measured from the base of that falsework element or system, and the assumed horizontal load will be applied to the falsework in accordance with the following:

- Actual loads due to equipment, construction sequence, or other causes, will be considered as acting at the point of application on the falsework element or system.
- Wind loads will be considered as acting at the centroid of the wind impact area for each height zone. When wind loads govern the design, however, the assumed horizontal load, used in calculating the overturning moment, is applied in a plane at the top of the falsework element or system. See Section 3-3.03, *Wind Loads*.
- When the minimum load governs the design, it is assumed as acting in a plane at the top of the falsework element or system.

6-5.04 Resisting Moments

When a horizontal load is applied to a falsework frame or tower, the overturning moment thus produced will be resisted up to a point by a resisting or righting moment generated by the weight of the total supported dead load. If the resisting moment is greater than the overturning moment, the falsework is stable against overturning and no external bracing will be required. If the resisting moment is less than the overturning moment, the difference must be resisted by bracing, guying cables, or other means of external support.

The resisting moments for falsework:

- In the unloaded condition may include the weight of falsework beams, forms and, reinforcing steel, but not the concrete.
- In the loaded condition may include the weight of falsework beams, forms, reinforcing steel, and the concrete.

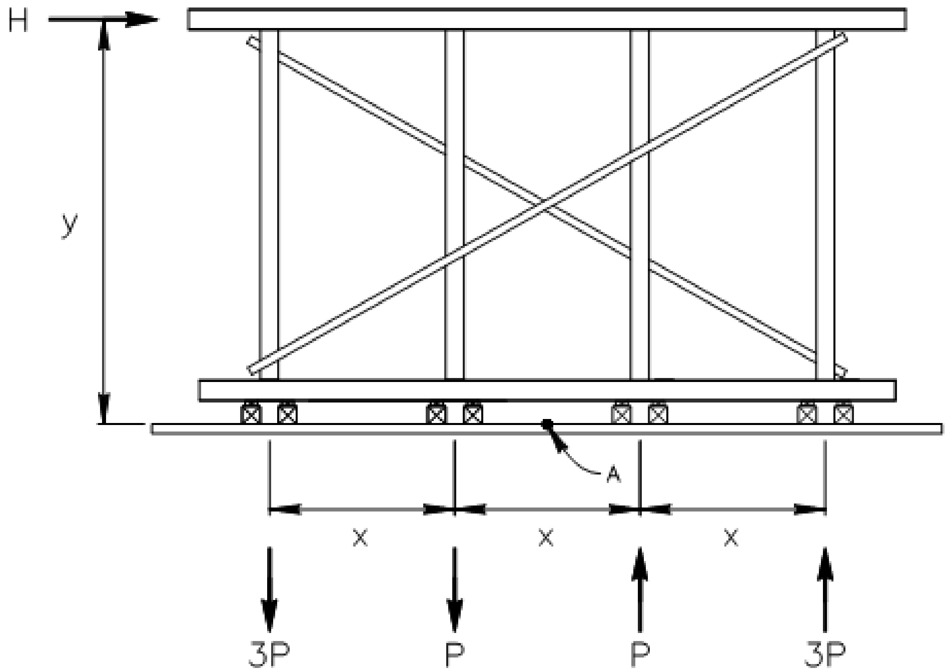
The weight of forms and rebar may be estimated, see Section 3-3.02, *Application*.

6-5.05 Effect of Overturning on Post Loads

When bracing is not required to resist overturning, do not overlook the effect of the overturning moment on post loads when the falsework is fully loaded (including the weight of concrete).

The post load (dead load plus live load) will be increased or decreased by the post reaction created by the overturning moment, or the vertical component of the resisting couple acting through the post. See Appendix D, *Example Problems*, Example 21, *Effect of Overturning on Post Loads.*

In a bent with rigid bracing and more than two posts, the post reactions are proportional to their distances from the center of rotation and may be obtained by algebraic summation, see Figure 6-6, *Effects of Overturning on Post Load*.



$$\Sigma M_A = Hy - [P(0.5x)+3P(1.5x)]2 = 0$$

$$= Hy - 10xP = 0$$

$$P = \frac{Hy}{10x}$$

Figure 6-6. Effects of Overturning on Post Load.

6-6 Tower Stability

6-6.01 Introduction

Falsework towers with discontinuous legs require additional analysis to ensure stability in the various stages of loading. The stages of loading are when the tower is unloaded, when loaded with falsework stringers, and during various loaded conditions involving concrete placement.

6-6.02 Stability

In addition to resisting collapse, the tower must be able to resist overturning and sliding at each plane that the tower is discontinuous.

Loaded towers will generally be capable of resisting overturning moments. However, unloaded towers, both during erection and during removal sequences, are vulnerable to overturning. Removal of portions of tower units while other portions are still loaded can lead to very unstable conditions. It is important to consider the effects of concrete pour sequences and the effects of the concrete weight in a span on one side of the tower and not the other.

Overturning stability of a tower is illustrated in Appendix D, *Example Problems,* Example 21, *Tower Stability*.

6-7 Pony Bent Systems

6-7.01 Introduction

Referring to Figure 6-7, *Pony Bent,* a bent stacked on top of another bent is typically called a pony bent. Pony bents are usually erected on, and supported by, a platform constructed at the top of the primary load carrying members. The platform functions as a horizontal diaphragm, and thus stabilizes the entire system. In this system, vertical load continuity and rotation of pony bent due to deflection of lower platform stringer must be accounted for in the design.

6-7.02 Stability

The stability of pony bent systems should be given special consideration. Pony bents should be independently braced, and the bracing must be capable of resisting the overturning moment produced by the assumed horizontal load acting at the top of the pony bent in both the transverse and longitudinal direction. Pony bents are often most vulnerable to overturning during erection and removal. Removal of portions of a pony bent while other portions are still loaded can lead to instability. There have been instances where falsework, which remained in place for an extended period of time collapsed, because the completed bridge deflected and redistributed loads to portions of the falsework.

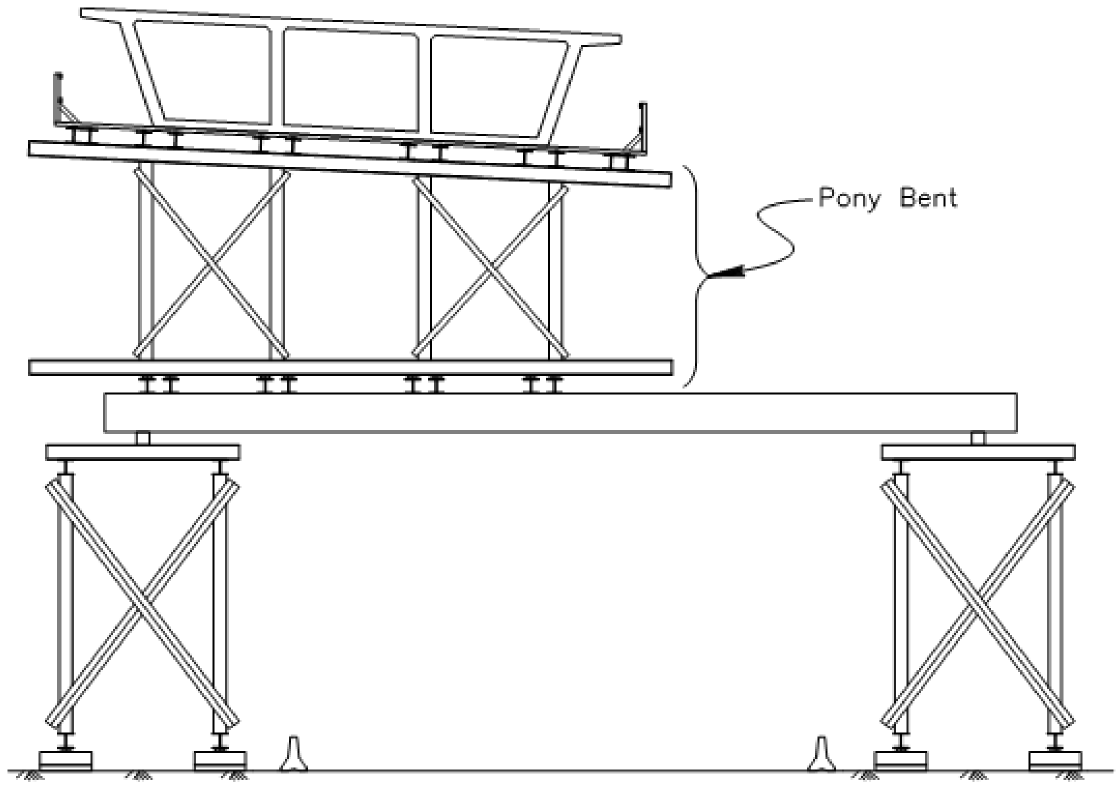


Figure 6-7 Pony Bent.

If a stabilizing platform is not incorporated into the falsework design, the individual bents must be braced or tied together in some manner to prevent lateral displacement at the bottom of the pony bent system.

6-8 Cap Systems

6-8.01 Introduction

In this section the term *cap* refers to top caps and bottom caps. Moreover, Figure 6-8, *Single Bottom Cap,* illustrates bottom caps, however, the requirements for top caps are similar.

The stability of the system will decrease as the distance between the supporting members and the top of the cap increases.

Single cap systems must adhere to the following:

- Maximum height to width ratio of 2:1 in any direction.
- Figure 6-8, *Single Bottom Cap*, shows the criterion for a bottom cap and prevention of overturning perpendicular to the centerline of the cap.

Stacked cap systems must also adhere to the following:

- Maximum height to width ratio of 2:1 in any direction.
- Stacked cap systems must be carefully designed and reviewed to verify stability.

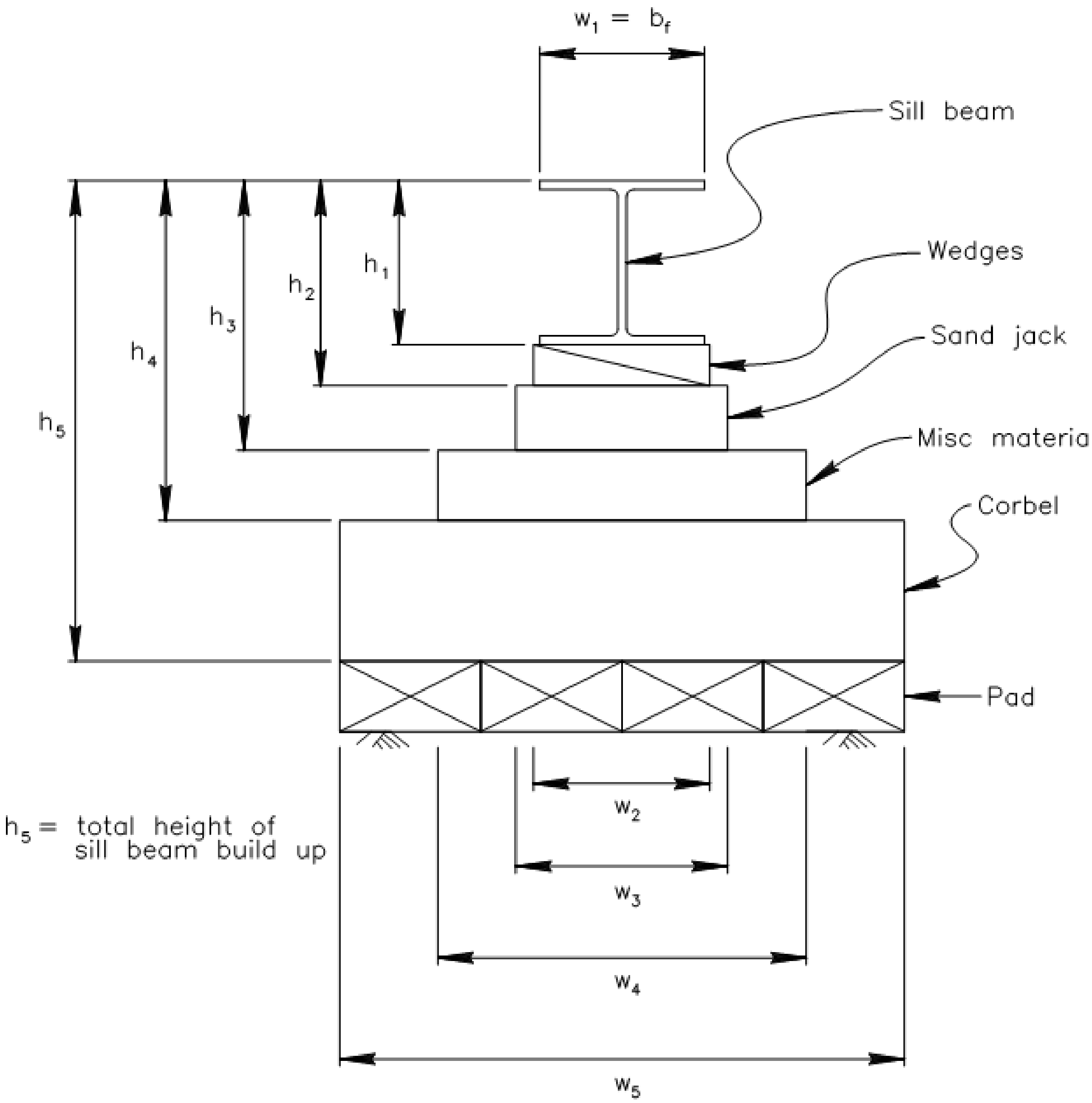
Multiple layers of supporting material must also adhere to a maximum height to width ratio, see Section 6-9, *Build-Up Material*.

The 2:1 height to width criteria must be strictly adhered to during both shop drawing review and construction. Using multiple caps or excessive stacking of material to correct grade errors discovered during falsework construction is an unacceptable construction practice and must not be allowed.

6-8.02 Analysis

The following must be considered when checking the shop drawings:

- All material above the pads or member set on the ground is included when checking the height to width ratio.
- Material size and thickness must be shown on the shop drawings.
- Material must have full bearing and be stacked tight and neat to provide uniform bearing for the supported members.
- Material placed on the sand jack plunger must have full bearing on the plywood plunger and must be clear of the frame of the sand jack by a minimum of 1/4-inch.
- If build-up material is used in the cap system, see section 6-9, *Build-Up Material,* for the height to width criteria.



To comply with 2:1 depth ratio, $h_i \leq 2w_i$ $(w_i > w_{(i-1)})$
(Overturning perpendicular to bent centerline)

Figure 6-8. Single Bottom Cap.

6-9 Build-Up Material

6-9.01 Introduction

The stability of build-up material decreases as the height of the stack increases. Build-up material must adhere to a maximum height to width ratio of 2:1 in all directions.

The 2:1 height to width criteria must be strictly adhered to during both shop drawing review and construction. Excessive stacking of material to correct grade errors discovered during falsework construction or to accommodate short posts is an unacceptable construction practice and is not allowed.

6-9.02 Analysis

The following must be considered when checking the shop drawings:

- All material above the pads or member set on the ground is included when checking the height to width ratio.
- Material size and thickness must be shown on the shop drawings.
- Material must have full bearing and be stacked tight and neat to provide uniform bearing for the supported members.
- Material placed on the sand jack plunger must have full bearing on the plywood plunger and must be clear of the frame of the sand jack by a minimum of 1/4-inch.
- If caps or stringers are supported by the build-up material, see section 6-8, *Cap Systems,* for the height to width criteria.

Chapter 7: Manufactured Assemblies

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7-1 Introduction

7-1.01 General Information

The term *manufactured assembly* means any commercial product, the use of which is governed by conditions and/or restrictions imposed by the manufacturer. Manufactured assemblies routinely used in falsework construction include products such as jacks, hangers, clips, brackets, and similar hardware products, as well as all types of manufactured shoring systems. When authorized for use, such products may be incorporated into the falsework design.

The *Standard Specifications*, Section 48-2.02B(3)(d), *Stresses, Loadings, and Deflections – Manufactured Assemblies,* limit the load on, and/or the deflection of, any commercial product to the maximum recommended or allowed by the manufacturer of the product.

Allowable loads and conditions or limitations of use must be shown in a catalog or technical manual published by the manufacturer, or in a certificate of compliance from the manufacturer pertaining to a particular project. If a certificate of compliance is furnished in lieu of catalog data, it must be shown on the manufacturer's shop drawing or included in a letter. For the statement to be accepted by Structure Construction (SC), it must be provided and signed by the manufacturer of the product under consideration, not by a material supplier or the contractor.

The *Standard Specifications*, Section 48-2.01C(1), *Falsework – Submittals – General,* require the contractor to furnish a written certification stating that all components of the assembly are used in accordance with the manufacturer's recommendations. Moreover, the contractor is required to furnish technical data for manufactured assemblies, when requested by the engineer. It is good practice for the engineer to request catalog data or other technical information, as it may be needed to verify the load carrying capacity of any manufactured product proposed for use in the falsework system.

Some manufacturer's catalogs are very brief and show only general details. The engineer must be able to verify that the item is used as the manufacturer intended. It is the responsibility of the engineer to contact the contractor to obtain the substantiating technical data or statement of compliance signed by the product manufacturer.

7-1.02 Contractor's Manufactured Assemblies

Noncommercial products or devices fabricated by contractors, also referred to as "contractor's manufactured assemblies," such as sand jacks and deck overhang brackets, may also be incorporated into the design. However, these assemblies must be tested per Section 7-2, *Load Tests,* before they can be authorized for use in falsework. The factor of safety is higher for noncommercial products or devices.

7-2 Load Tests

7-2.01 Load Tests Introduction

In any case where the shop drawings show or describe a manufactured product or device, which cannot be found in any catalog, a load test is required to establish the safe load carrying capacity of that product or device.

Load testing a commercially available product or device is intended to determine or verify the load carrying capacity of the product or device. Load testing must not be used to establish that a particular design detail or method of construction is capable of withstanding the imposed load without failing if the design calculations show that it is overstressed. Similarly, load testing must not be used to establish that the manufacturer’s recommended load can be exceeded nor that the device can be used in a different manner than recommended by the manufacturer.

If a tested device is to be used on more than one job, the contractor must inform the structure representative of the intent to use a tested device. The structure representative must verify the submitted information by comparing it to the testing records in SC HQ.

7-2.01A Safe Working Load and Factor of Safety

Devices can be tested to a predetermined value or to failure. It is recommended to test the device to failure, in which case the safe working load may be taken as 1/2 the ultimate load. This will provide an **FS** = **2** with respect to failure, which is consistent with manufacturer's ratings for concrete form accessories. If it is not possible to test to failure, the working load used for the design should not exceed 1/2 of the maximum load carried during the test.

All testing of commercially available metal shoring systems must include a minimum **FS = 2.5**. See also Section 7-2.03, *Metal Shoring Systems,* and 7-4, *Metal Shoring Systems*.

The procedure discussed herein for load testing of manufactured assemblies will also apply to noncommercial items, such as noncommercial deck overhang brackets, fabricated by the contractor (i.e. contractor’s manufactured assemblies). If the product is noncommercial, **FS = 3** is required, except for sand jacks, see Section 7-2.02, *Sand Jacks*.

7-2.01B Procedure

The load test can be performed in the contractor's yard or at the job site. The load test should be performed under conditions which will closely simulate the intended field use, particularly as to the method of support. The load test must be set up so that it accurately represents the use of the product. When performing a test of a manufactured assembly, follow these steps:

- Notify the engineer and the SC Falsework Engineer two weeks prior to performing testing.
- Before performing the test, the contractor must submit a testing procedure including shop drawings of the assembly, sketches of the testing equipment and setup, explaining what is being tested, how much load it is being tested for, and how the setup simulates field conditions.
- Before performing the test, the contractor must submit the annual certification record of the testing equipment performed by an independent third party.
- The testing procedure must be submitted to the Falsework Engineer in SC HQ for review and authorization. When authorized, the contractor may perform the test.
- The contractor must write a report of the test including the testing procedure, testing equipment certification, drawings of the tested assembly, test results, and photos. This report must be stamped by a civil engineer registered in the State of California and submitted to the Falsework Engineer in SC HQ.
- The structure representative and the Falsework Engineer or their delegates must witness the test and document the test procedure and results. These notes must be submitted to the Falsework Engineer in SC HQ.

Testing should, as a minimum, comply with the following:

- Test at least three identical assemblies or devices.
- The set up must include other necessary features unique to the system being tested.
- Measure strain, stresses, deflection, and other values as necessary to properly determine the adequacy of the device or as deemed necessary by the engineer.
- Apply load incrementally and hold load for one minute before taking readings. 10% incremental loading is recommended.
- Apply cyclic loading when deemed necessary by the engineer. It is recommended to use at least 10 cycles.
- The allowable load is the average of the test results divided by the factor of safety.

- Submit a statement certifying that all components duplicate field use of the proposed devices, components, or systems.

7-2.02 Sand Jacks

Sand jacks must be tested per Section 7-2, *Load Tests.* In addition, the specific requirement listed below must be satisfied:

- Hold the design load for 20 minutes with less than 1/16-inch increase in vertical displacement.
- Test the sand jack assemblies to twice the design load with less than 1-inch vertical displacement of the plunger. This provides an FS = 2.

7-2.03 Metal Shoring Systems

All commercially available metal shoring systems must be tested per Section 7-2, *Load Tests.* In addition, the specific requirement listed below must be satisfied:

- All testing must be performed by the manufacturer.
- Contractors may not perform their own testing on metal shoring systems.
- The test must demonstrate the adequacy of the proposed assemblies in resisting the design load (including the assumed horizontal load).

Past load test results, performed by the original manufacturer, that are compatible with the conditions of use may be acceptable as supporting documentation of the load carrying capacity of the shoring system. The test results must demonstrate the adequacy of the proposed assemblies in resisting the design load including the assumed horizontal load. Moreover, these test results must be sent to the Falsework Engineer in SC HQ for review and authorization.

The safe working loads for some older shoring systems previously used on Caltrans jobs were determined empirically from full scale load tests. The *Scaffolding, Shoring, and Forming Institute* and *Canadian Standards Association* provide recommendations for testing and rating shoring towers and components. The rating included a minimum **FS = 2.5**. In all cases of record, maximum values were obtained during tests under ideal conditions, in which the legs of the test tower were loaded uniformly and concentrically, and the tower was supported on a concrete pad to ensure an unyielding foundation. Results of tests in which the towers were loaded eccentrically, and/or lateral movement was allowed indicate a substantial reduction in capacity.

Noncommercial metal shoring systems may not be used.

7-3 Miscellaneous Manufactured Assemblies

7-3.01 Wood Sand Jacks

When wood sand jacks are used, the contractor has the following two options:

- Construct and use the authorized wood sand jacks. See Section 7-3.01A, *Authorized Wood Sand Jack.*
- Construct sand jacks that deviate from the authorized wood sand jacks and test them per Section 7-2, *Load Tests,* and 7-2.02, *Sand Jacks.*

Sand jacks must be new and manufactured for the current job. It is not acceptable to reuse old wood sand jacks.

Figure 7-1. Wood Sand Jacks.

7-3.01A Authorized Wood Sand Jack

The sand jack shown in Figure 7-2, *Authorized Wood Sand Jack,* is authorized for use on jobs in California. The allowable load is 68 kips and the anticipated settlement is 1/2-inch.

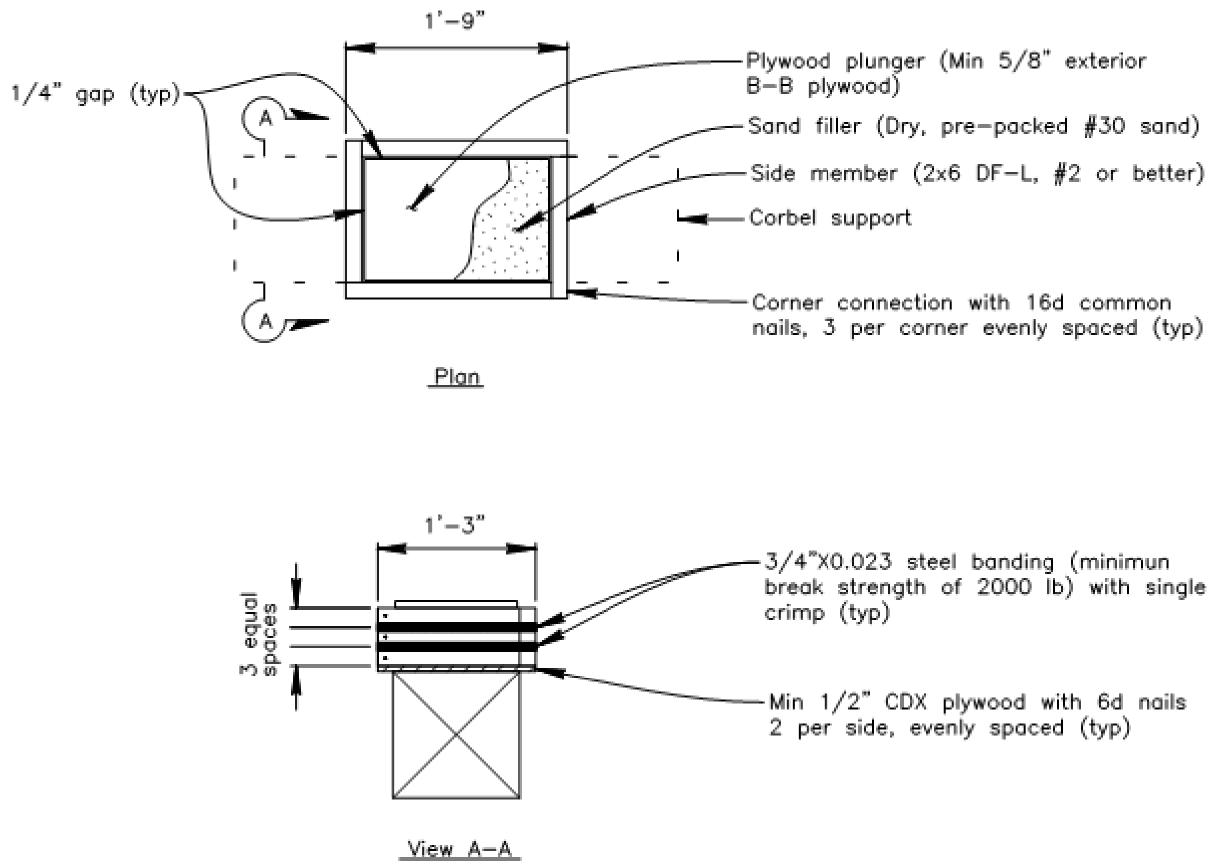


Figure 7-2. Authorized Wood Sand Jack.

7-3.02 Steel Sand Jacks

Steel sand jacks must be tested per Section 7-2, *Load Tests,* and 7-2.02, *Sand Jacks*. Steel sand jacks may be reused on other projects in the same configuration as they were tested if authorized by the structure representative. The shop drawing submitted for the steel sand jacks must be the same as those used for the test.

Figure 7-3. Steel Sand Jack.

7-3.03 Beam Hangers

Beam hangers are hardware items which are placed transversely across the top flange of a beam or girder. Threaded rods or bolts are supported by single or double channels, wire loops, or other methods as per the manufacturer. The rods or bolts hang vertically or diagonally to support deck falsework or overhang brackets. Figure 7-4, *Beam Hangers,* shows a generic beam hanger.

Unbalanced loading (loading only one side of the hanger) will reduce the load-carrying capacity of the hanger unless it is of a type designed to be loaded on one side at a time, or unless special measures are taken to hold the hanger in place. Special measures may include welding to studs or shear connectors. Beam hangers must not be welded to the top flange of a steel girder or to precast-prestressed girder stirrups.

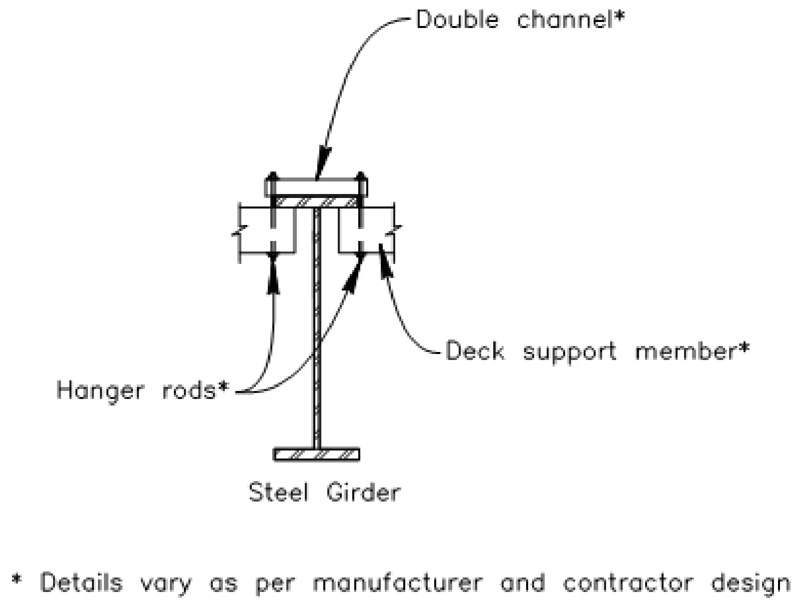


Figure 7-4. Beam Hangers.

7-3.04 Deck Overhang Brackets

Several types of commercial and noncommercial metal brackets specifically designed to support cantilevered deck overhangs are available. On some brackets the diagonal leg is wood. On precast concrete (PC) girders, these brackets are typically supported by beam hangers or by form bolt inserts cast into the top of the PC girder stems. On steel girders these brackets are typically supported by threaded rods or bolts extending through holes drilled in the web of steel girders. The brackets typically have a diagonal leg braced against the bottom flange of the girder.

The *Standard Specifications*, Section 55-1.03B, *Steel Structure – Construction – Falsework,* governing steel construction includes certain restrictions affecting the design of falsework supporting deck overhangs on steel girder bridges. See also Section 4-11, *Concrete Deck on Steel Girders*.

Bracket on PC girder **Bracket on steel girder**

Figure 7-5. Deck Overhang Brackets.

7-3.05 Metal Joist Assemblies

Metal joist assemblies are essentially metal beams, which can be adjusted to provide a wide range of span lengths. Catalog data should be provided showing the safe load carrying capacity and the allowable deflection. When joist assemblies are used to support deck slabs between girders, design load deflection is limited to the maximum deflection allowed by the manufacturer, which in some cases may exceed 1/240 of the span. At all other locations, the limit in the Standard Specification Section 48-2.02B(3)(c), *Design Criteria – Stresses, Loadings, and Deflections – Steel,* applies.

Figure 7-6. Metal Joists.

7-3.06 C-Clamps

Heavy-duty commercial or non-commercial C-clamps (Figure 7-7, C-*Clamps*) having a torque-tightening capacity of 90 ft-lb or more may be used as connecting devices in accordance with the criteria in this section.

Commercial **Non-Commercial**

Figure 7-7. C-Clamps.

C-clamps used in conjunction with angles clamped to beams as show in Figure 7-8, *C-Clamp Installation*, will be permitted for transmitting forces in accordance with the criteria in this section. Other configurations must be tested per Section 7-2, *Load Tests*.

7-3.06A Commercial C-Clamps

Commercially available C-clamps must conform to the following:

- Heavy duty service pattern clamps with not less than a 10,000 lb. load limit (generally drop forged premium quality steel).
- Must remain elastic while withstanding a torque of 90 ft-lb load on the bolt.
- Bolt must be hardened machine bolt with cupped tip with a 3/4-inch diameter or greater.
- The contractor must furnish a catalog cut or manufacturer's technical data sheet describing the clamp in sufficient detail to verify compliance with product criteria listed in this section.

7-3.06B Non-Commercial C-Clamps

Non-commercial C-clamps must conform to the following:

- Clamps must be Grade A36 steel or higher.
- Must remain elastic while withstanding a torque of 90 ft-lb load on the bolt.
- Bolt must be hardened machine bolt with cupped tip with a 3/4-inch diameter.
- Dimension and details must be as shown in Figure 7-9, *Non-Commercial C-Clamp.*
- The shop drawings must include a sketch showing the dimensions and details of the clamp. The clamp must comply with restrictions and requirements in this section.

7-3.06C Working Load

The C-Clamp working loads stated below are based on test results:

- When used as shown in Figure 7-8, *C-Clamp Installation*, the two-clamp configuration can resist a maximum of 6000 lbs.
- When used without an angle iron, single C-clamps may be used as a mechanical connection at traffic openings for stringer to top cap connection to resist at most a 500 lb. load in any direction including uplift.

7-3.06D Use and Installation

The use and installation of C-clamps must be in conformance with the following:

- The location of the clamps must be shown on the shop drawings.
- Clamps must be torqued to 90 ft-lb. The shop drawings must include a note requiring all clamps to be torqued to a minimum of this value.
- All flanges, angle legs, plates, etc. to be connected must have constant thickness.
- Beams and caps must not be clamped together to resist longitudinal forces
- Clamps used to connect steel stringers to steel caps are to be placed on the heaviest loaded span side of the stringers.
- Must not be installed on the tail end of beams or stringers, but on the span side.
- May only be used for one purpose per installation, for example a clamp used to resist a 500 lb. force may not be used to resist other forces.

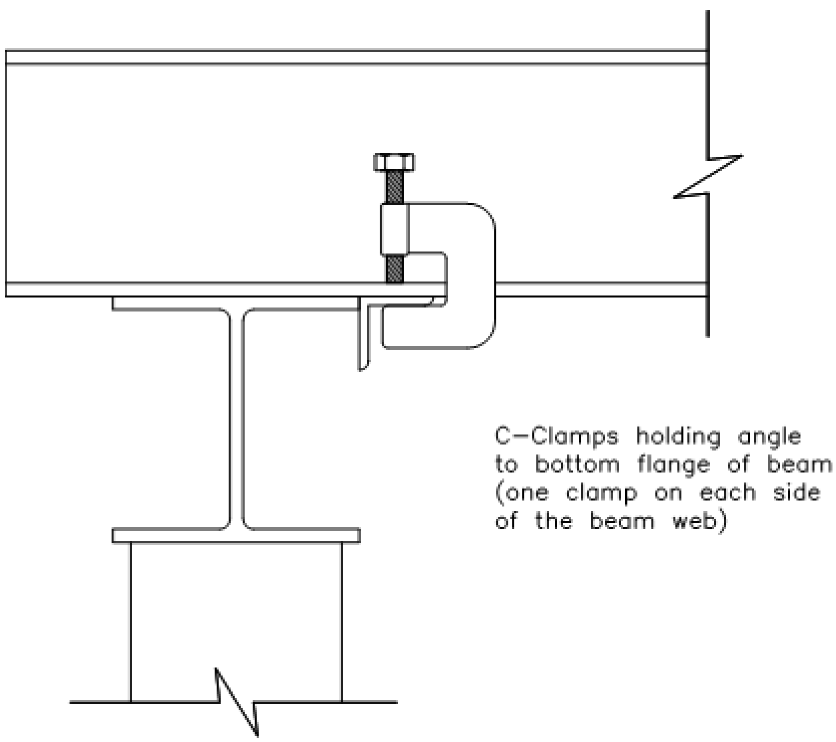
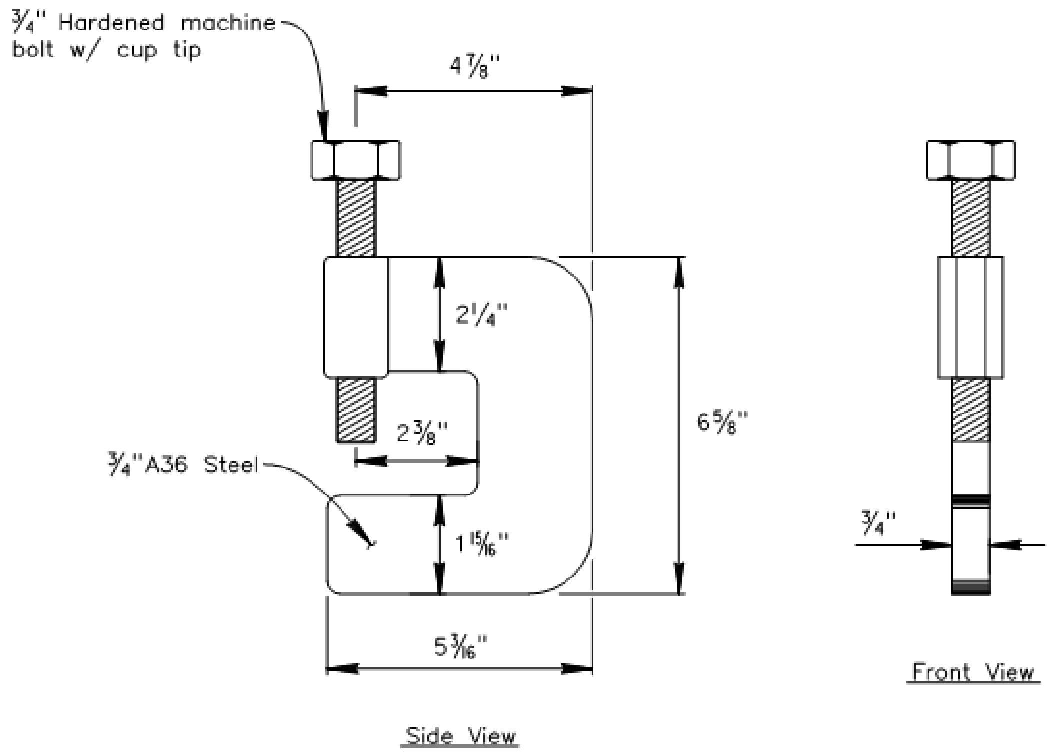


Figure 7-8. C-Clamp Installation.



Side View **Front View**

Figure 7-9. Non-Commercial C-Clamp.

7-3.07 Stringer Connector

Stringer connectors are simple non-commercial connection devices used to transmit longitudinal forces along the length of falsework stringers.

Each stringer connector consists of two 1/2-inch thick A36 steel plates approximately 6-inches long by 2.50-inches wide, bolted at one end with a 3.5-inches long 7/8-inch diameter ASTM F3125, Grade A325 bolt. The lower plate is U-shaped, and the upper plate has a rectangular hole for the banding with a rounded edge where the banding attaches to the plate. The clips are typically installed on stringer top flanges with the lower plate clamping the stringer web, the bolt butting the end of the stringer, and the upper plate parallel to the banding. The banding is looped through the holes in the upper plates connecting the two clips. Figure 7-10, *Stringer Connector Details,* shows the connectors and method of installation on the stringers.

These stringer connectors have been tested and authorized for use in falsework to resist or transfer longitudinal forces in stringers.

7-3.07A Working Load

The stringer connector working loads are based on the stringer connectors shown in Figure 7-10, *Stringer Connector Details,* with a single 1.25 inches wide by 0.035" thick band. Higher loads will not be permitted. The approved working loads are:

- 5000 lbs. when the angle between center lines of banding and beam webs does not exceed 30°.
- For larger angles decrease the working load value by 1700 lbs. for each l0° increment in excess of 30°.

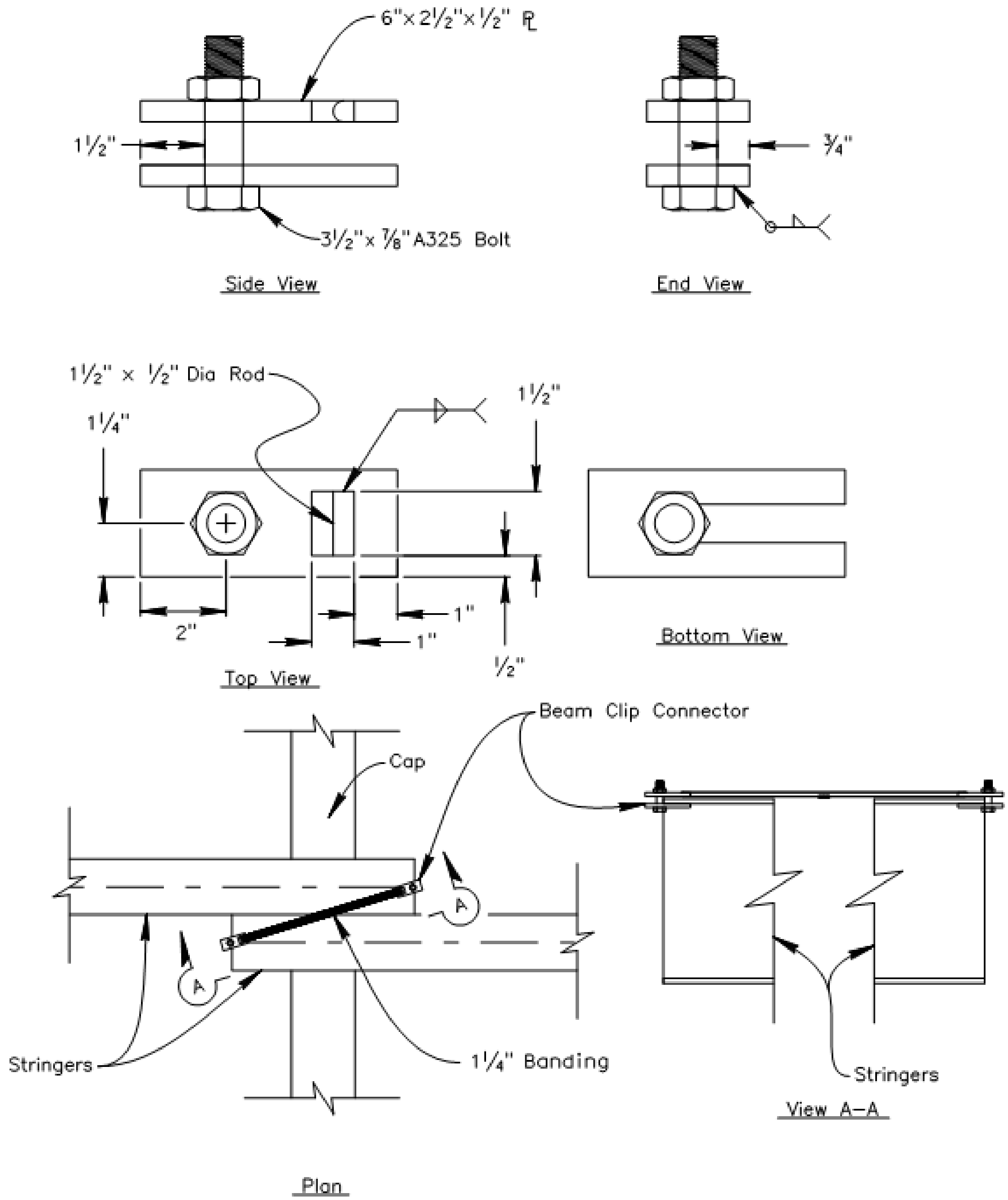


Figure 7-10. Stringer Connector Details.

7-3.08 Pile Friction Collars

Typically, the friction collar is used to permit erection of falsework on the friction collars before the piles for flat slab bridges are cut to grade. The falsework will normally be set on sand jacks which will be mounted atop the friction collar brackets. Friction collars must be installed per the manufacturer's recommendations.

Figure 7-11. Pile Friction Collar.

7-3.09 Beam Clips

Beam clips that conform to Figure 7-12, *Beam Clip,* and are fastened with a minimum of five 20D nails can be used to resist loads defined in *Standard Specifications*, Section 48-2.02B(4), *Special Locations*. Beam clips not conforming to Figure 7-12, *Beam Clip,* must be tested per Section 7-2, *Load Tests*.

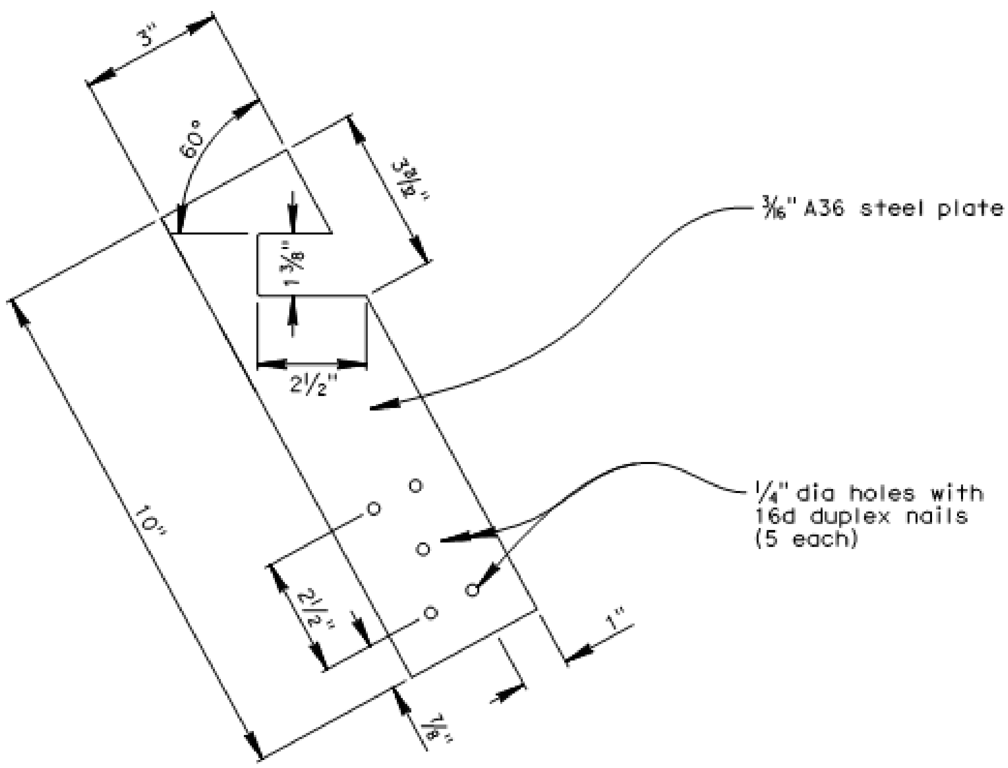


Figure 7-12. Beam Clip.

7-4 Metal Shoring Systems

7-4.01 Introduction

This section describes the general criteria and procedures to be followed when designing and reviewing metal shoring systems for compliance with the falsework specifications.

The term *metal shoring system* describes falsework consisting of individual components that may be assembled and erected in place to form a series of internally braced metal towers of any desired height. The tower legs, directly, or through a cap system, support the main load carrying members and transmit the load to a stable foundation. The use of metal shoring systems will require a complete submittal, review, and authorization process. The term *metal shoring system* includes all necessary bracing to stabilize the system and the cap beam connecting shoring towers.

Shoring towers are indeterminate space frames and therefore cannot be analyzed by the general formulas applicable to statically determinate framed structures. Manufacturers use empirical criteria developed from the effects of tower height, differential leg loading, side sway, and method of external support to determine the ability of metal shoring to safely carry a given load. Three-dimensional computer modeling is often used to design the system and full-scale load tests are performed to confirm the modeling. From this the manufacturer establishes the safe working loads. When used in projects, the contractor may be required to provide three-dimensional computer analysis (including electronic files) as part of the falsework submittal.

Figure 7-13. Heavy Duty Shoring System.

In the past, some proprietary metal shoring systems were analyzed and approved for use on state jobs. However, past approvals for these metal shoring systems are no longer valid. See Section 7-5, *Previously Approved Metal Shoring Systems*.

7-4.02 Safe Working Load and Factor of Safety

A metal shoring system is considered a manufactured assembly. Therefore, the maximum load to be carried must not exceed the safe working load recommended by the manufacturer for any given loading condition, see *Standard Specifications*, Section 48-2.02B(3)(d), *Stresses, Loadings, and Deflections – Manufactured Assemblies*.

The safe working load provided by the manufacturer must have a minimum **FS = 2.5**.

The shoring capacity, as shown in catalogs or brochures published by the manufacturer, should be considered as the maximum allowable safe working load that the shoring is able to safely support under ideal loading conditions. These maximum values must be reduced for adverse loading conditions often encountered in bridge falsework. For example, horizontal loads, eccentricity due to unbalanced spans or pouring sequence, and uneven foundation settlement are but a few of the loading conditions typical of bridge falsework which differ from the loading conditions upon which the manufacturer's ratings are based.

Finally, the maximum allowable safe working load as recommended by the manufacturer is based on the use of new material or used material in good condition. Shoring components which are not in good condition will not be allowed for use in metal shoring systems.

7-4.02A Testing

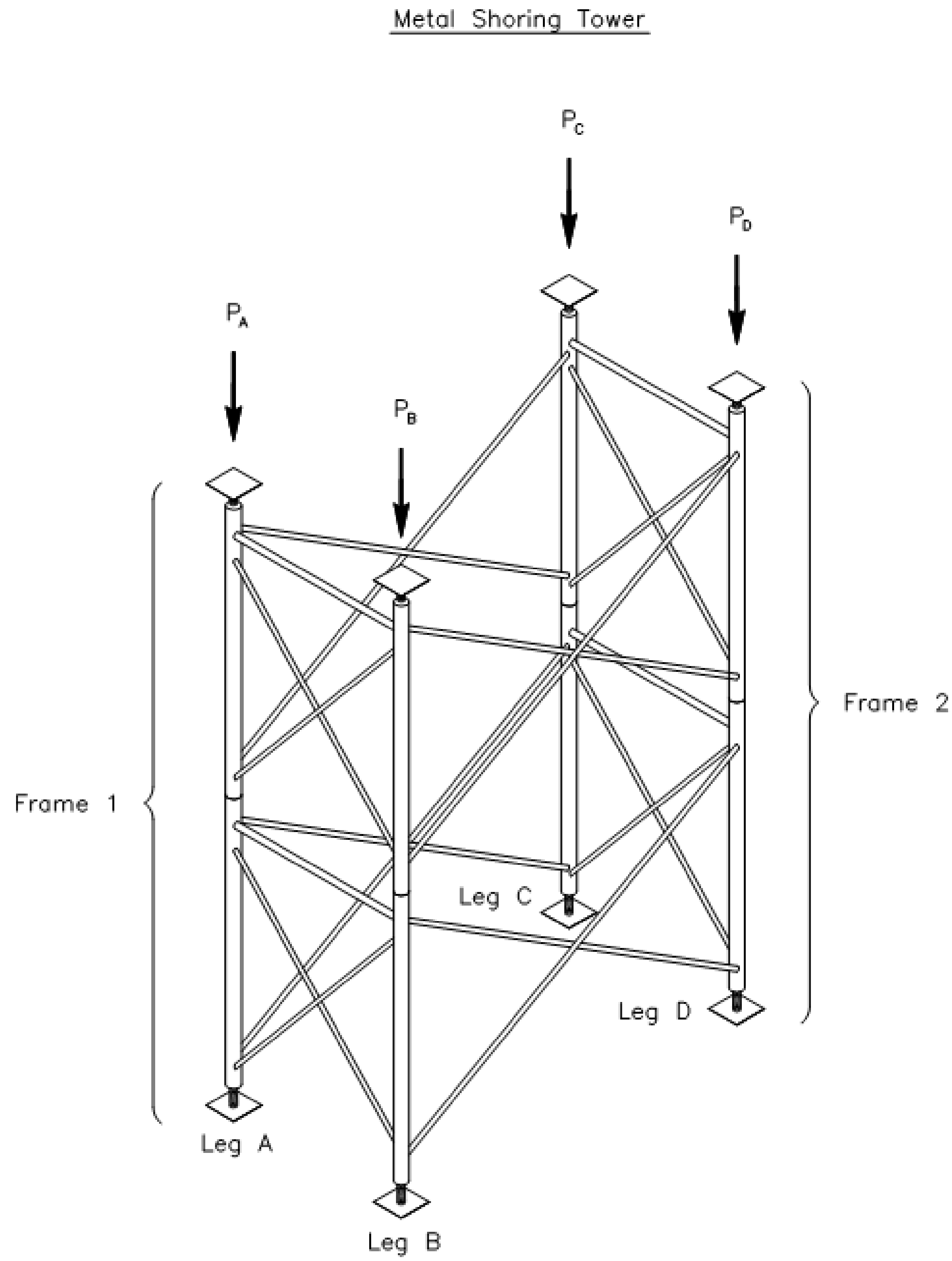
For testing of metal shoring systems, see Sections 7-2, *Load Tests,* and 7-2.03, *Metal Shoring Systems*.

7-4.03 Design Criteria

Metal shoring systems must comply with all manufacturer's recommendations for condition, use, assembly, loading, foundation, and all other restrictions. Under no circumstances must the manufacturer's design criteria be exceeded. Metal shoring systems must be designed to meet the following criteria:

- Metal shoring systems must be independently designed by a civil engineer registered in the State of California. This includes the entire shoring system with bracing for overturning and collapse and cap beams between shoring towers or units.
- The manufacturer's recommended safe working loads must be adjusted to consider material, site, and loading conditions. The maximum safe working load must be reduced for adverse loading conditions often encountered in bridge falsework.
- The shoring system must resist the sum of the dead loads and live loads and an assumed horizontal load in accordance with the requirements in Chapter 3, *Loads,* and *Standard Specifications*, Section 48-2.02B(2), *Design Criteria – Loads.*
- Steel beams used as continuous caps over two or more tower units require a complete stress analysis to determine the effect of continuity on tower leg loads. In many cases, particularly where large skews are involved, the falsework stringers will not be supported directly over a tower leg, therefore, both positive and negative bending moments will occur in the cap. Resulting moment and shear must be added to or subtracted from the simple beam reaction to obtain the actual leg load, and this may produce a significant load differential.
- The effect on the stress in the cap from differential leg loads, the resulting differential leg shortening, and differential foundation settlement must be accounted for.

- The maximum load on one leg of a tower should not exceed 4 times the load on any other leg under any given loading condition or sequence. The maximum load on one of the two frames making up a tower should not exceed 4 times the load on the opposite frame under any given loading condition or sequence. See Figure 7-14, *Leg Load in Shoring Tower.*
- Bracing must be provided to prevent overturning, collapse, and lateral deflection at the top of the shoring system.
- When investigating the stability of a multiple tower bent, any advantage gained from the theoretical transfer of the point of application of the vertical forces as the towers start to tip must be neglected.
- In a multiple tower bent where a continuous cap is not used, each tower must be independently braced to resist the horizontal load applied to that tower because the supported falsework above the towers is not capable of transferring horizontal forces.
- The foundation must be designed to ensure that the vertical loads are uniformly distributed and differential settlements are minimized.
- For individual shoring towers with maximum leg loads exceeding 30 kips, the foundation must be designed to provide uniform settlement under all legs of the tower.
- Timber pads or cribbing, while generally adequate for conventional falsework construction, may not be adequate to minimize differential settlement and ensure uniform settlement under metal shoring systems. Tower manufacturers will generally recommend concrete to ensure an unyielding foundation. Under adverse foundation conditions, cast-in-drilled-hole piles may provide the best value solution.
- Elastic shortening of the aluminum posts must be included in net settlement considerations.
- Wind loads on towers may be computed as outlined in Section 3-3.03, *Wind Loads.*
- Comply with *Standard Specifications*, Section 48-2.02B(4), *Design Criteria – Special Locations,* when constructed at the applicable locations.
- Metal shoring systems that have been altered must not be used. If a job specific alteration is necessary, the alteration must be designed by a civil engineer registered in the State of California.



Maximum Leg Load in Tower: $P_A \leq 4P_{B,\ C\ or\ D}$

Maximum Frame Load in Tower: $P_A + P_B \leq 4(P_C + P_D)$

Figure 7-14. Leg Load in Shoring Tower.

7-4.04 Submittal

Metal shoring system submittals must include:

- Shop drawings in accordance with the *Standard Specifications*, Section 48-2.01C(2), *Submittals – Shop Drawings*. Shop drawings must be sealed and signed by the designer and the checker.
- Design calculations sealed and signed by the designer.
- Check calculations sealed and signed by the checker.
- Manufacturer's data to verify the manufacturer's recommendations, including safe working loads and corresponding factor of safety.
- The bracing system, including all construction details, must be shown on the shop drawings.
- Limits of screw jack extensions and the corresponding load rating must be shown on the shop drawings.
- Statement signed by the shoring manufacturer covering the specific installation. The statement must expressly provide that the shoring will carry the loads to be imposed without overstressing any component of the metal shoring system or reducing the required factor of safety. The statement is a condition for authorization of the falsework design. If the contractor cannot or does not furnish the statement, the shop drawings must not be authorized.

Upon request by the engineer, perform and provide test data demonstrating the adequacy of the proposed assemblies in resisting the design load (including the assumed horizontal load). Past load test results that are compatible with the conditions of use may be acceptable. Refer to Section 7-2.03, *Metal Shoring Systems,* for testing requirements.

Upon request by the engineer, perform and provide results from rigorous three-dimensional analysis demonstrating the adequacy of the proposed assemblies in resisting the design load (including the assumed horizontal load). Past analysis that are compatible with the conditions of use may be acceptable. Provide electronic calculations and models of the analysis.

Prior to concrete placement, A letter of certification that certifies all components of the metal shoring system are used in compliance with the manufacturer's recommendations, in accordance with the *Standard Specifications*, Section 48-2.01C(1), *Falsework – Submittals – General.*

7-4.05 Review of Shop Drawings

Review the metal shoring system shop drawings according to the requirements in Chapter 2, *Review of Shop Drawings*. Verify that the metal shoring system complies with Section 7-4.03, *Design Criteria*.

The structure representative or the civil engineer registered in the State of California reviewing the submittal must consult with the Falsework Engineer in the SC HQ when reviewing metal shoring systems.

Similar systems may be load rated differently depending upon the bracing type used in the frame. For example, towers with ladder type frames typically have a much lower rating than similar towers with cross braced frames.

The extension of screw jacks affects the load rating of shoring systems.

7-4.06 Letters of Certification

The *Standard Specifications*, Section 48-2.01C(1), *Falsework – Submittals – General,* requires the contractor to submit a letter of certification, which certifies that all components of the manufactured assembly are used in accordance with the manufacturer's instructions. In addition, the *Standard Specifications*, Section 48-2.01C(2), *Falsework – Submittals – Shop Drawings,* and the Cal-OSHA Construction Safety Order §1717(c)(1), *Falsework and Vertical Shoring – Inspection,* require another certification, which certifies that the falsework (which the shoring is part of) is constructed as shown in the authorized shop drawings before concrete is placed.

7-4.07 Field Inspection

Chapter 9, *Inspection* provides the guidelines for the field inspection of metal shoring systems.

7-5 Previously Approved Metal Shoring Systems

In the past, some proprietary metal shoring systems were analyzed and approved for use on state jobs. However, past approvals for these metal shoring systems are no longer valid for the following reasons:

- The proprietary metal shoring systems are often leased to many contractors for multiple types of applications and the conditions of the metal shoring components may not be the same as when they were first analyzed.
- Liability issues as ownership of those proprietary systems may have changed, especially when those reviews were done many years ago.
- Material specifications have changed in some instances and SC has not been informed of the changes.

Information about these shoring systems is available from the Falsework Engineer in SC HQ and can be used as a reference for when reviewing similar shoring systems.

7-6 Concrete Pads

7-6.01 Introduction

Concrete pads may be used as an alternative to timber pads. When concrete pads are used, the contractor has the following two options:

- Construct and use the authorized concrete pads per Section 7-6.02, *Authorized Concrete Pads.*
- Design and construct concrete pads that deviate from the authorized concrete pads per Section 7-6.03, *Other Concrete Pads*:

7-6.02 Authorized Concrete Pads

The pads shown in Figure 7-15, *Authorized Concrete Pad,* are authorized for use on projects in California. 12 x 12 timber corbels may be used, but it is more common to use steel beam corbels, which are anchored into the concrete. Both timber and steel corbels must be installed as shown in Figure 7-15, *Authorized Concrete Pad*. Lifting cables or bars are commonly installed in the concrete.

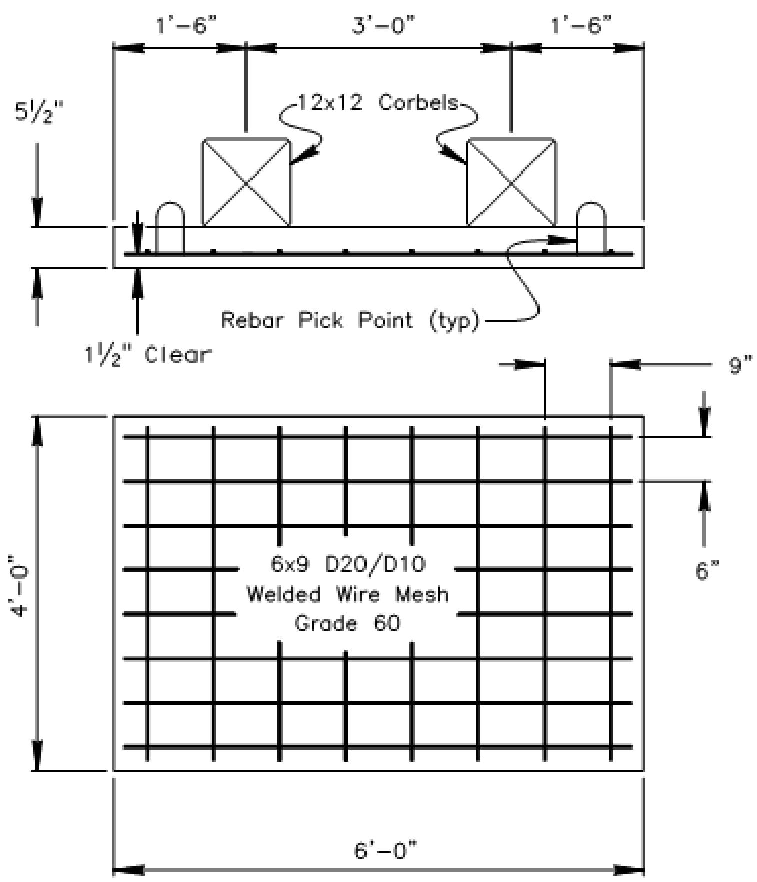


Figure 7-15. Authorized Concrete Pad.

7-6.02A Working Load

These pads are designed for these maximum loads:

- Soil pressure of 4000 psf
- 100 kips post load with two corbels supporting the post load as shown in Figure 7-16, *Concrete Pads with Cap Beam*.

Figure 7-16. Concrete Pads with Cap Beam.

7-6.02B Design Criteria

Design and analysis were completed using text book load factor design (LFD) for concrete. The concrete pads meet the criteria of wood pads as outlined in Section 8-2.04D, *Pad Analysis at Exterior Post*.

The pads must comply with the following requirements:

- Dimensions: 6 feet long by 4 feet wide by 5.5 inches thick.
- Both timber and steel corbels must be installed as shown in Figure 7-15, *Authorized Concrete Pad*. The location of the corbel is critical to the flexural strength of the pad.
- The corbels must extend across the full width of the pad.
- Concrete strength must not be less than **f'c** = 3500 psi.
- The reinforcing welded wire mesh must be D20/D10 Grade 60, spaced at 6 inches transversely and 9 inches longitudinally.
- The longitudinal bars in the welded wire mesh must be located at the bottom of the mesh.
- The clearance to the longitudinal reinforcing bars in the mesh from the bottom of the pad must be 1.5 inches.

7-6.02C Certificate of Compliance

A certificate of compliance from the pad fabricator must be obtained for concrete pads to be used on the project. The certificate of compliance must:

- Certify that the concrete meets the compressive strength requirements.
- Certify that the steel mesh is of the type and quality specified.
- Certify that the pad is fabricated as indicated in Figure 7-15, *Authorized Concrete Pad.*
- Be stamped by a civil engineer registered in the State of California.
- State how the individual pads can be identified in the field.

7-6.03 Other Concrete Pads

If the contractor chooses not to use the authorized concrete pads, the contractor may design and fabricate other concrete pads. These pads must comply with the requirements in the following sections.

7-6.03A Design Criteria

The design of the pads and corbels must comply with these requirements:

- The pads must be designed and independently checked by two civil engineers registered in the State of California.
- The pads must be designed using the latest AASHTO Bridge Design Specifications for concrete pads.
- Timber corbels must be designed using the current *Falsework Manual*.
- Steel corbels must be designed using the latest AASHTO Bridge Design Specifications or latest AISC manual for steel.
- Submit the design details, design calculations, and check calculations to the Falsework Engineer in SC HQ for review and authorization for use in California.
- The design must include the maximum allowable soil pressure, the maximum allowable post load, and the required bearing area between the post and corbel.

7-6.03B Testing

The Falsework Engineer in SC HQ will determine if testing of the pads is required for authorization, in accordance with Section 7-2, *Load Tests*.

7-6.03C Certificate of Compliance

Once the pads are authorized for use in California, a certificate of compliance from the pad fabricator must be obtained for concrete pads to be used on the project. The certificate of compliance must:

- Certify that the concrete meets the design compressive strength requirements.
- Certify that the steel mesh is of the type and quality specified.

- Certify that the pad is fabricated as per the submitted design and details. The details of the pads and the authorization will be available from the Falsework Engineer in SC HQ.
- Be stamped by a civil engineer registered in the State of California.
- State how the individual pads can be identified in the field.

Chapter 8: Foundations

Table of Contents

8-1 Introduction

This chapter discusses the methods and procedures used by Structure Construction (SC) to evaluate the adequacy of falsework pad and pile foundations.

To a certain extent, the procedures are approximations, which were developed from a subjective evaluation of the manner in which falsework pads and piles react when loads are applied. Although empirical in some cases, the procedures give results that are acceptable in the light of falsework requirements. To ensure uniformity, procedures are to be followed by bridge field personnel in all cases when reviewing the contractor's design for structural adequacy and compliance with contract requirements.

From an administrative standpoint, the elements of the falsework system comprising the foundation differ from other elements of the system in one important aspect. The Standard Specifications, Section 48-2.03B, *Temporary Structures – Falsework – Construction - Foundations,* permit the contractor to place falsework pads and drive falsework piles before the design has been reviewed and the shop drawings authorized. Pad placement and pile driving must be inspected to the extent necessary to ensure adequate foundation support at the time the work is done. Any inconsistencies and differences between the shop drawings and the work being performed in the field should be brought to the contractor's attention immediately.

8-2 Timber Pads

8-2.01 Introduction

Individual posts may be supported by individual pads, which may be square or rectangular. A row of several posts may be supported by a continuous pad. Falsework pads may consist of a single member or of several members set side by side. Normally, for continuous pads, a lower cap beam is used to distribute load from the posts to the corbels.

Corbels are short beams which are used to distribute the post load or lower cap load across the top of the pads. In a conventional falsework bent with 12 x 12 timber posts, the corbel is usually a timber member of the same dimensions as the post.

When the vertical design load is very high, as is often the case for a falsework bent adjacent to a wide traffic opening or under the long hinge side, it is often necessary to use two or more closely spaced corbels to adequately distribute the load over the pad. Steel beams are also often used as corbels at locations where post loads are relatively high.

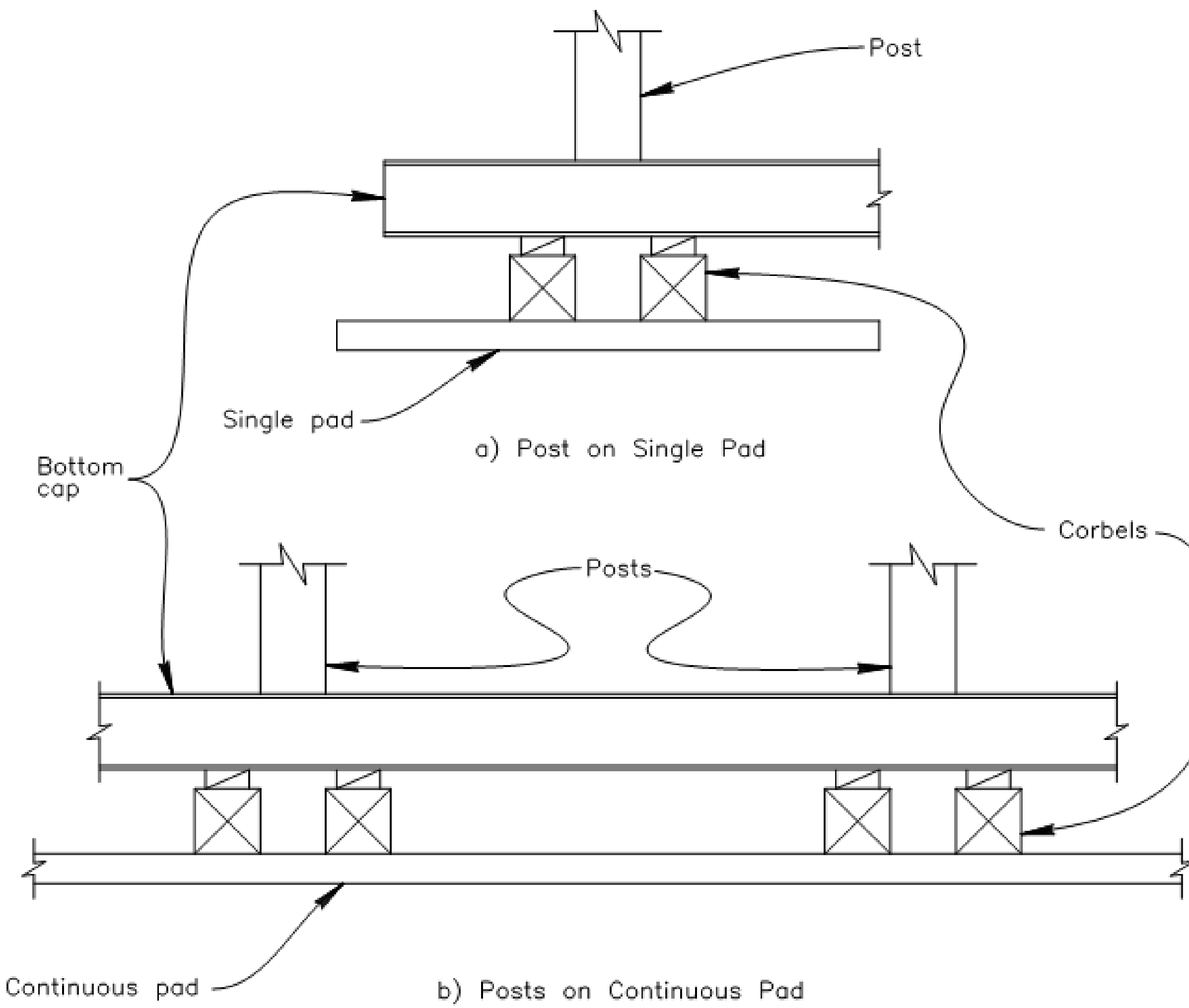


Figure 8-1. Timber Pads.

As a general design procedure, a pad may be viewed as a cantilever beam extending from the face of the post or corbel. With the beam loaded uniformly with the soil pressure, bending and shear stresses may be calculated. However, this approach will not give exact values because the assumed uniform load distribution does not occur in practice.

To facilitate analysis of timber pad systems, SC has developed an empirical procedure, which provides sufficient pad rigidity to assure a reasonably uniform load distribution. The procedure is explained in the following sections and illustrated in Appendix D *Example Problems* in Example 19 and 20, *Individual Falsework Pads, and* Example 21 and 22, *Continuous Pads.*

8-2.02 Effective Bearing Length of Continuous Pad Systems

The effective bearing lengths given by the SYM formula in Section 8-2.02A, *Effective Bearing Length for Uniform Post Spacing (SYM Formula,)* is the pad length where the bending stress in the pad equals the allowable bending stress and is the maximum length over which a pad is theoretically capable of distributing the post load uniformly. However, the pad length is limited to physical constraints, such as the post or corbel spacing.

Since the formula is based on bending, it is not necessary to calculate the bending stress, because for a given post load, any pad length less than the length given by the formula will produce a bending stress that is less than the allowable stress.

8-2.02A Effective Bearing Length for Uniform Post Spacing (SYM Formula)

Referring to Figure 8-2, *Theoretical Effective Length for Uniform Post Spacing,* in a continuous pad system where the posts are uniformly spaced, the effective bearing length of the pad, measured in the direction of the wood grain, is equal to the 1/2 post width plus 2 times the length of a cantilever extending from a point midway between the center and edge of the post or corbels to a distance such that the calculated bending stress in the pad equals the allowable stress.

$$\mathbf{L_e = \frac{t}{2} + 2L_c} \qquad \text{(8-2.02A-1)}$$

where $\mathbf{L_e}$ = Effective bearing length of pad (ft)

$\mathbf{t}$ = Width of post or corbel (ft)

$\mathbf{L_c}$ = Length of cantilever extending from a point midway between the center and edge of the post or corbel to a distance such that the bending stress in the pad is zero (ft)

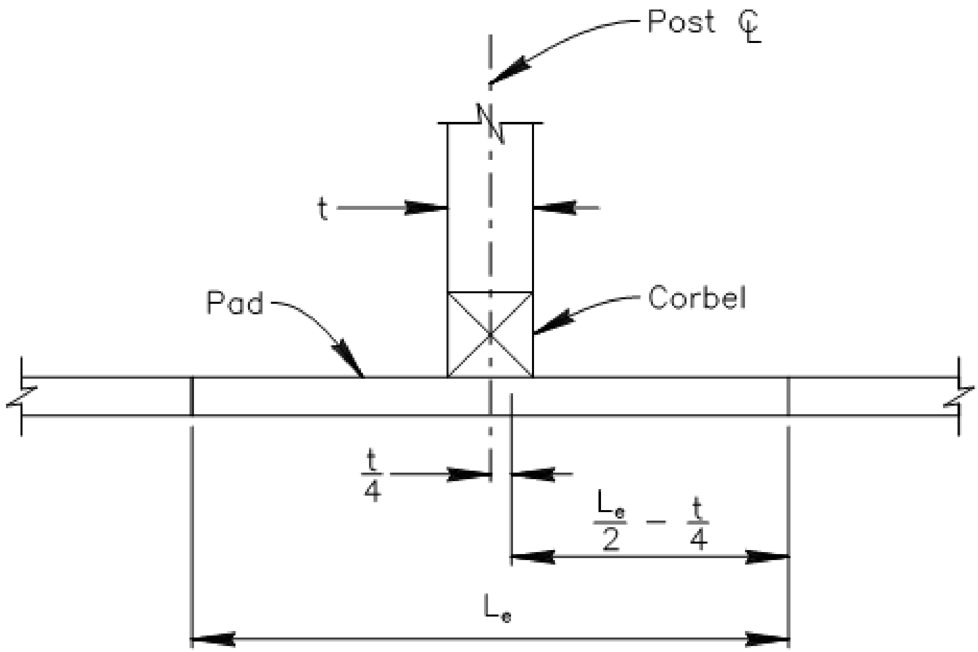


Figure 8-2. Theoretical Effective Length for Uniform Post Spacing.

The formula below can be used to calculate the effective length at an interior post when the post spacing is uniform along the pad.

The equation was derived from a simplified method. The simplified formula gives results that are accurate within 3% for the range of post loads and member sizes commonly used for falsework construction in California. For descriptive purposes, the simplified formula is designated the "SYM" formula.

The SYM formula is:

$$\mathbf{L_{SYM}} = \frac{1}{12}\left(\frac{8F_b{}'S}{1000P} + t\right) \qquad \text{(8-2.02A-2)}$$

where $\mathbf{L_{SYM}}$ = Effective length of pad (ft)

$\mathbf{F_b{}'}$ = Adjusted bending stress design value (psi)

S = Section modulus of pad (in^3)

P = Post load (kips)

t = Width of post or corbel (in)

The pad bearing length is determined by:

$$\mathbf{L_b = \text{ smaller of} \begin{cases} \text{post spacing} \\ L_{SYM} \end{cases}} \quad \text{(8-2.02A-3)}$$

8-2.02B Effective Bearing Length for Non-Uniform Post Spacing

Referring to Figure 8-3, *Pad Length for Non-Uniform Post Spacing*, in a continuous pad system where the posts are not uniformly spaced, the pad is asymmetrical for analysis. For the asymmetrical condition, the limiting pad length on one side of a post does not always equal the limiting length on the opposite side, and the two respective lengths must be determined independently.

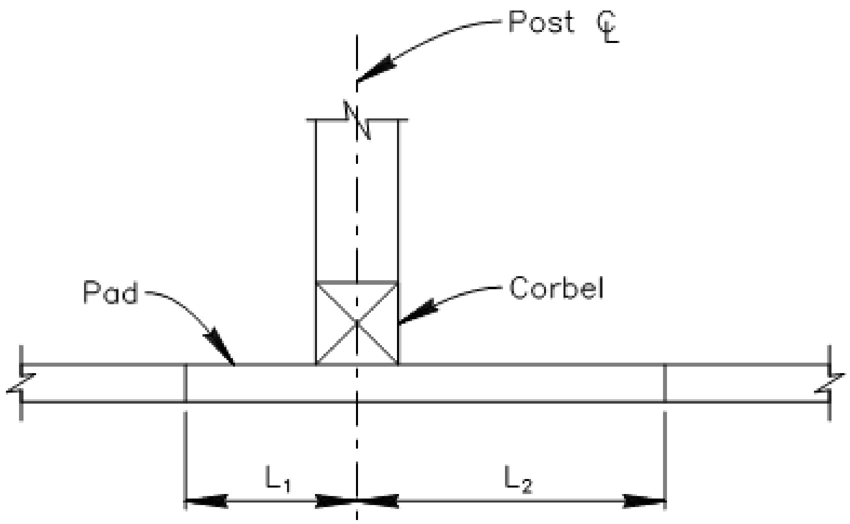


Figure 8-3. Pad Length for Non-Uniform Post Spacing.

The pad length on each side is determined by the SYM formula discussed in Section 8-2.02A, *Effective Bearing Length for Uniform Post Spacing (SYM Formula)*.

The limiting pad bearing length on the short side, $\mathbf{L_1}$, is determined by:

$$\mathbf{L_1 = \text{ smaller of} \begin{cases} \frac{1}{2}\text{post spacing on the short side} \\ \frac{1}{2}L_{SYM}, \ \text{see equation 8-2.02A-2} \end{cases}} \quad \text{(8-2.02B-1)}$$

The limiting pad bearing length on the long side, $\mathbf{L_2}$, is determined by:

$$\mathbf{L_2 = \text{ smaller of} \begin{cases} \frac{1}{2}\text{post spacing on the long side} \\ \frac{1}{2}L_{SYM}, \ \text{see equation 8-2.02A-2} \end{cases}} \quad \text{(8-2.02B-2)}$$

The total pad bearing length is the sum of the limiting values above and will be discussed in more detail in Section 8-2.04, *Continuous Pad with Single Corbel,* and Section 8-2.05, *Continuous Pad with Two or More Corbels*.

8-2.03 Soil Bearing Pressure Under Continuous Pad

The soil bearing pressure under the continuous pad within the area of the bearing length is calculated by:

$$\boldsymbol{\sigma_b} = \frac{\mathbf{1000P}}{\mathbf{L_b}\left(\frac{\mathbf{b}}{\mathbf{12}}\right)} \qquad \text{(8-2.03-1)}$$

where $\boldsymbol{\sigma_b}$ = Soil bearing pressure (psf)

$\mathbf{L_b}$ = Total bearing length of the pad (ft)

P = Post load (kips)

b = Width of pad (in)

8-2.04 Continuous Pad with Single Corbel

This section shows the procedure for continuous timber pads with load distribution by a single corbel per post. Procedures are shown for uniformly and non-uniformly spaced interior posts and at an exterior post.

8-2.04A Horizontal Shear Stress in Pads

The equations for the horizontal shear stress, $\mathbf{f_v}$, consider the pad as a continuous beam loaded uniformly with the soil pressure beyond the distance, **d**, from the post or corbel, where **d** is the pad thickness. See Section 5-2.04C, *Horizontal Shear*, for additional information about horizontal shear.

8-2.04A(1) Uniform Post Spacing (Symmetrical Analysis)

The horizontal shear stress in a continuous pad with uniformly spaced posts is determined by:

$$\mathbf{f_v} = \left(\frac{3}{2}\right)\frac{\left\{\dfrac{1000P\left[\frac{L_b}{2} - \frac{\frac{t}{2}}{12} - \frac{d}{12}\right]}{L_b}\right\}}{bd} \qquad \text{(8-2.04A(1)-1)}$$

where $\mathbf{f_v}$ = Horizontal shear stress in the pad on the long side (psi)

P = Post load (kips)

L_b = Total bearing length of the pad (ft)

t = Width of post or corbel (in)

d = Thickness of pad (in)

b = Width of pad (in)

8-2.04A(2) Non-Uniform Post Spacing (Asymmetrical Analysis)

The horizontal shear stress on the long side of the post in a continuous pad with non-uniformly spaced posts is determined by:

$$\mathbf{f_v} = \left(\frac{3}{2}\right)\frac{\left\{\dfrac{1000P\left[L_2 - \dfrac{\frac{t}{2}}{12} - \dfrac{d}{12}\right]}{L_b}\right\}}{bd} \qquad \text{(8-2.04A(2)-1)}$$

where f_v = Horizontal shear stress in the pad on the long side (psi)

P = Post load (kips)

L_2 = Pad length on long side (ft)

t = Width of post or corbel (in)

d = Thickness of pad (in)

L_b = Total bearing length of the pad (ft)

b = Width of pad (in)

8-2.04B Pad Analysis at Interior Post with Uniform Spacing

Figure 8-4, *Pad at Interior Posts with Uniform Spacing,* shows a falsework bent where the post spacing (PS) is uniform along a continuous pad and the post load is distributed across the pad by a single corbel.

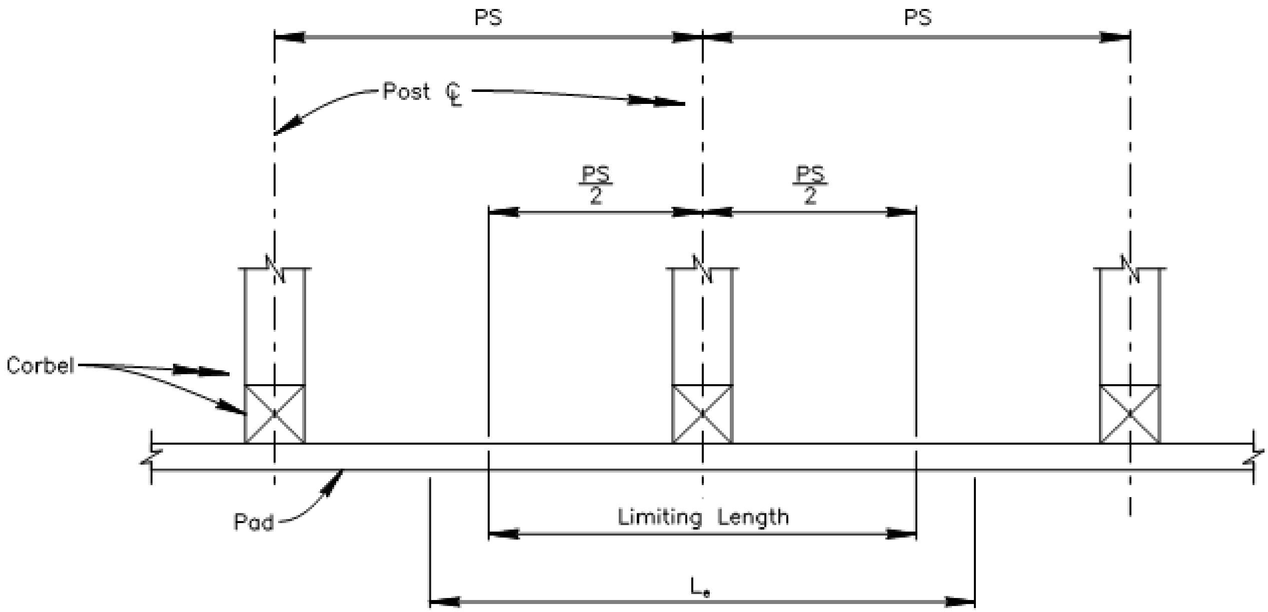


Figure 8-4. Pad at Interior Posts with Uniform Spacing.

When the post spacing is uniform, the bearing length is symmetrical. Analyze the pad as follows:

1. Calculate the effective length, $\mathbf{L_e}$, of the pad using the SYM formula:

$$\mathbf{L_e = L_{SYM}} \text{ from equation 8-2.02A-2} \qquad \text{(8-2.04B-1)}$$

2. The limiting bearing length, $\mathbf{L_b}$, is determined by:

$$\mathbf{L_b = \text{smaller of} \begin{cases} \text{Post Spacing} \\ L_e \end{cases}} \qquad \text{(8-2.04B-2)}$$

3. Calculate the soil pressure, σ_b, using the limiting bearing length, $\mathbf{L_b}$, from step 2 and compare to the allowable soil bearing value. Use the equation 8-2.03-1 for σ_b.

4. Calculate the pad stress, $\mathbf{f_v}$, due to horizontal shear using the limiting bearing length, $\mathbf{L_b}$, from step 2. Calculate the stress at a distance, **d**, from the face of the post or corbel where **d** is the pad thickness. Use equation 8-2.04A(1)-1 for $\mathbf{f_v}$.

8-2.04C Pad Analysis at Interior Post with Non-Uniform Spacing

Figure 8-5, *Pad at Interior Posts with Non-Uniform Spacing,* shows a falsework bent where the post spacing is non-uniform along a continuous pad and the post load is distributed across the pad by a single corbel. When the post spacing is not uniform, the contribution to system adequacy made by the pad on one side of a post must be determined independently of the contribution made by the pad on the opposite side.

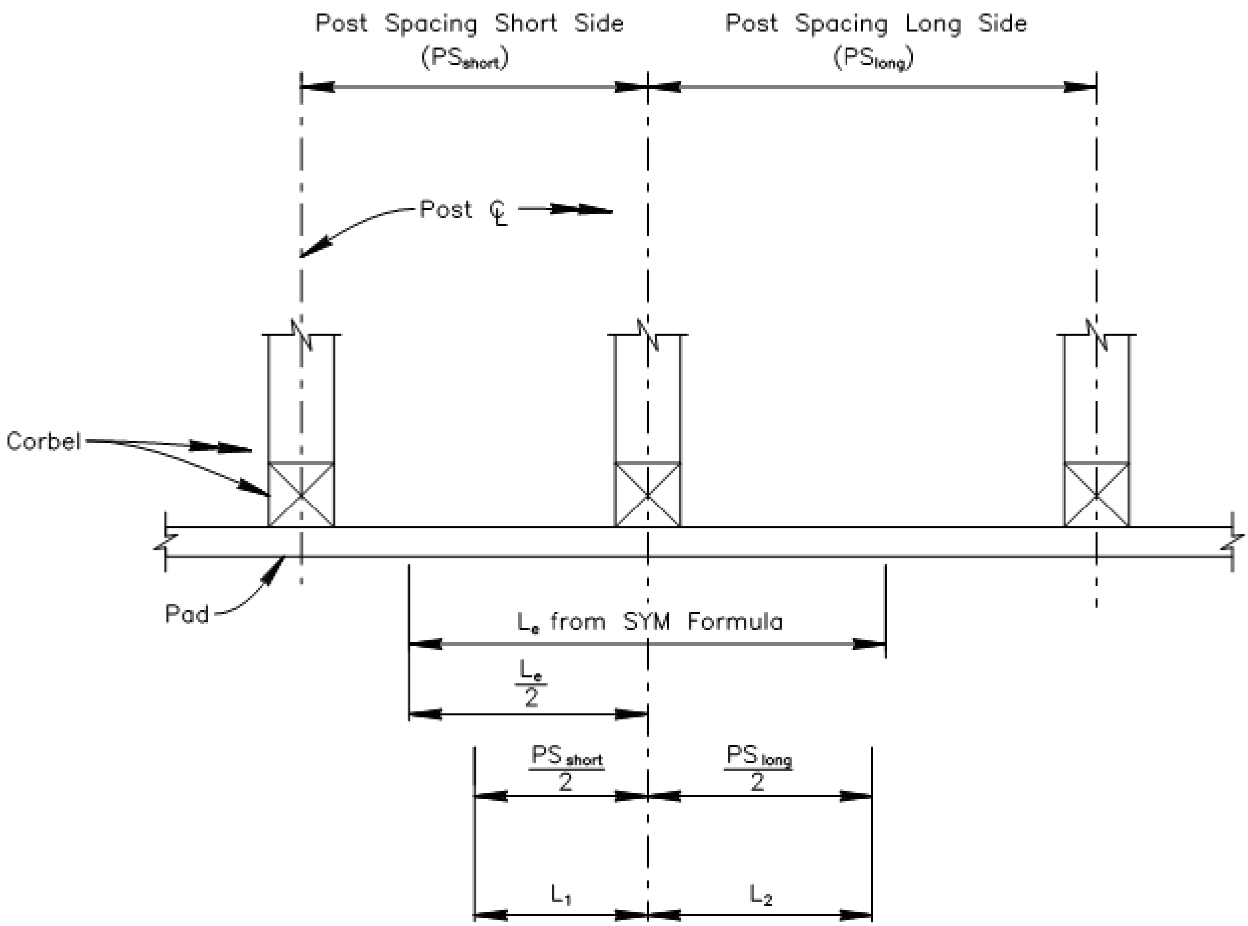


Figure 8-5. Pad at Interior Posts with Non-Uniform Spacing.

When the post spacing is non-uniform, the bearing length can be asymmetrical or symmetrical. Begin with the side that has the shorter post spacing. In this case it is the left side. Analyze the pad as follows:

1. Calculate the effective length, $\mathbf{L_e}$, of the pad using the SYM formula:

$$\mathbf{L_e} = \mathbf{L_{SYM}} \text{ from equation 8-2.02A-2} \qquad (8\text{-}2.04C\text{-}1)$$

2. The limiting bearing length on the short side, $\mathbf{L_1}$, is determined by:

$$\mathbf{L_1 = \text{smaller of} \begin{cases} \frac{PS_{short}}{2} \\ \frac{L_e}{2} \end{cases}} \quad \text{(8-2.04C-2)}$$

3. The limiting bearing length on the long side, $\mathbf{L_2}$, is determined by:

$$\mathbf{L_2 = \text{smaller of} \begin{cases} \frac{PS_{long}}{2} \\ \frac{L_e}{2} \end{cases}} \quad \text{(8-2.04C-3)}$$

4. Asymmetrical Analysis: If $\mathbf{L_1 \neq L_2}$, the bearing length is asymmetrical. (If the lengths are equal, skip to step 5)

   a. The bearing length, $\mathbf{L_b}$, is determined by:

$$\mathbf{L_b = L_1 + L_2} \quad \text{(8-2.04C-4)}$$

   b. Calculate the soil pressure, $\boldsymbol{\sigma_b}$, using the limiting bearing length, $\mathbf{L_b}$, from step 4a and compare to the allowable soil bearing value. Use the equation (8-2.03-1) for $\boldsymbol{\sigma_b}$.

   c. Calculate the pad stress, $\mathbf{f_v}$, on the long side due to horizontal shear using the lengths $\mathbf{L_2}$ from step 3 and $\mathbf{L_b}$ from step 4a. Calculate the stress at a distance, **d**, from the face of the post or corbel where **d** is the pad thickness. Use equation (8-2.04A(2)-1) for $\mathbf{f_v}$.

5. Symmetrical Analysis: If $\mathbf{L_1 = L_2}$, the bearing length is symmetrical.

   a. The limiting bearing length, $\mathbf{L_b}$, is the length determined in step 1:

$$\mathbf{L_b = L_e} \quad \text{(8-2.04C-5)}$$

   b. Calculate the soil pressure, $\boldsymbol{\sigma_b}$, using the limiting bearing length, $\mathbf{L_b}$, from step 4a and compare to the allowable soil bearing value. Use the equation (8-2.03-1) for $\boldsymbol{\sigma_b}$.

   c. Calculate the pad stress, $\mathbf{f_v}$, due to horizontal shear using the limiting bearing length, $\mathbf{L_b}$, from step 4a. Calculate the stress at a distance, **d**, from the face of the post or corbel where **d** is the pad thickness. Use equation 8-2.04A(1)-1 for $\mathbf{f_v}$.

8-2.04D Pad Analysis at Exterior Post

This section shows the procedures for timber pads at exterior posts. Figure 8-6, *Pad at Exterior Post,* shows a falsework bent with an exterior post on a continuous pad and the post load is distributed across the pad by a single corbel.

For exterior posts, the contribution to the system adequacy made by the length of pad on the outside of the post must be determined independently of the contribution made by the pad on the inside.

For typical bent configurations and post spacing, the pad length on the inside of the post will be the long side for the analysis. However, the procedure is also valid in any case where the long side length is on the outside.

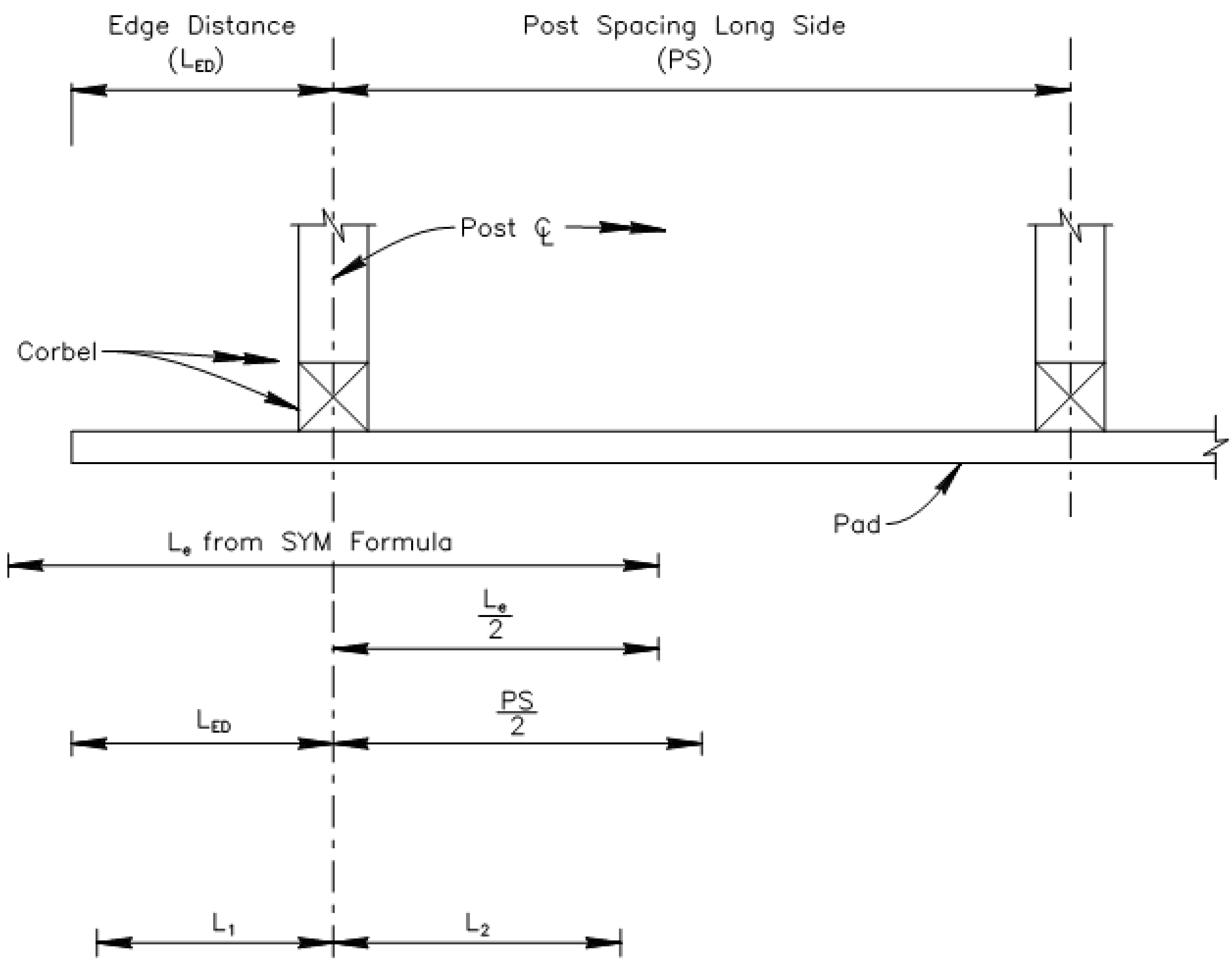


Figure 8-6. Pad at Exterior Post.

For exterior posts the bearing length can be asymmetrical or symmetrical. Begin with the outside, in this case the left side. Analyze the pad as follows:

1. Calculate the effective length, $\mathbf{L_e}$, of the pad using the SYM formula:

$$\mathbf{L_e = L_{SYM}} \text{ from equation 8-2.02A-2} \qquad (8\text{-}2.04D\text{-}1)$$

2. The limiting bearing length on the outside, $\mathbf{L_1}$, is determined by:

$$\mathbf{L_1 = \text{smaller of}} \begin{cases} \mathbf{L_{ED}} \\ \mathbf{\frac{L_e}{2}} \end{cases} \qquad (8\text{-}2.04D\text{-}2)$$

3. The limiting length on the inside, $\mathbf{L_2}$, is determined by:

$$\mathbf{L_2 = \text{smaller of}} \begin{cases} \mathbf{\frac{PS}{2}} \\ \mathbf{\frac{L_e}{2}} \end{cases} \qquad (8\text{-}2.04D\text{-}3)$$

4. Asymmetrical Analysis: If $\mathbf{L_1 \neq L_2}$, the bearing length is asymmetrical. (If the lengths are equal, skip to step 5).

   a. The bearing length, $\mathbf{L_b}$, is determine by:

   $$\mathbf{L_b = L_1 + L_2} \qquad (8\text{-}2.04D\text{-}4)$$

   b. Calculate the soil pressure, $\boldsymbol{\sigma_b}$, using the limiting bearing length, $\mathbf{L_b}$, from step 4a and compare to the allowable soil bearing value. Use the equation 8-2.03-1 for $\boldsymbol{\sigma_b}$.

   c. Calculate the pad stress, $\mathbf{f_v}$, on the long side due to horizontal shear using the greater of L1 and L2 from steps 2 and 3 and $\mathbf{L_b}$ from step 4a. Calculate the stress at a distance, **d**, from the face of the post or corbel where **d** is the pad thickness. Use equation 8-2.04A(2)-1 for $\mathbf{f_v}$.

5. Symmetrical Analysis: If $\mathbf{L_1 = L_2}$, the bearing length is symmetrical. This is unlikely to occur in actual practice:

   a. The limiting bearing length, $\mathbf{L_b}$, at the exterior post under consideration is the sum of the lengths from step 3 and step 4.

$$\mathbf{L_b = L_1 + L_2} \quad \text{(8-2.04D-5)}$$

b. Calculate the soil pressure, $\boldsymbol{\sigma_b}$, using the limiting bearing length, $\mathbf{L_b}$, from step 5a and compare to the allowable soil bearing value. Use the equation 8-2.03-1 for $\boldsymbol{\sigma_b}$.

c. Calculate the stress in the pad, $\mathbf{f_v}$, due to horizontal shear using the limiting bearing length, $\mathbf{L_b}$, from step 5a. Calculate the stress at a distance, **d**, from the face of the post or corbel where **d** is the pad thickness. Use equation 8-2.04A(1)-1 for $\mathbf{f_v}$.

8-2.05 Continuous Pad with Two or More Corbels

It is common to use a falsework system where the post load is transferred through a lower cap to two or more corbels as shown in Figure 8-7, *Pad with Double Corbels*. This section discusses a procedure on how to analyze continuous pads in double corbel systems. This procedure should be used when the clear distance between adjacent corbels is equal to or less than twice the pad thickness.

In cases where more than two corbels are used, the length, **m**, is the distance measured centerline-to-centerline between the two outermost corbels in the system.

In the preceding sections, the term post spacing has been used in the procedure for continuous pads with one corbel per post. However, in the double corbel system, the continuous pad analysis considers the corbel spacing rather than the post spacing.

In some cases, the load from two or more posts will contribute to the total vertical load to be distributed through the corbel system. For this configuration, the total load applied to the system must be used to calculate the effective length of the pad.

When the vertical load is distributed to a continuous pad through a system of two closely spaced corbels, the pad distributes the load as though it were imposed by a single corbel having a width along the pad of approximately the distance between the outside faces of the adjacent corbels. Because of this behavior, the procedure discussed in the preceding sections would give limiting lengths that are shorter, and soil bearing values that are higher, than is the case. Therefore, SC developed the following procedure for systems with two or more corbels.

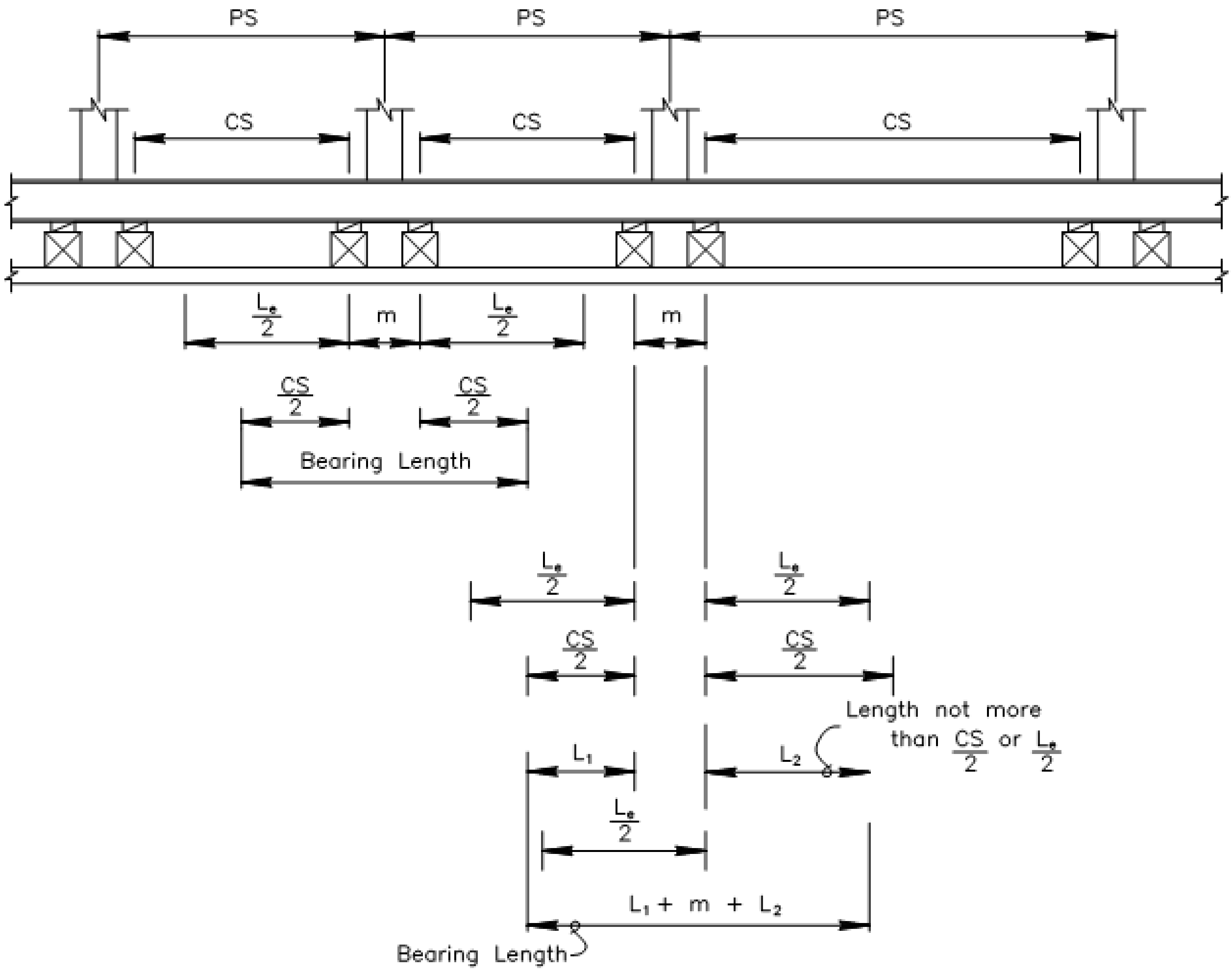


Figure 8-7. Pad with Double Corbels.

8-2.05A Horizontal Shear Stress in Pads

The equations for the horizontal shear stress, $\mathbf{f_v}$, consider the pad as a continuous beam loaded uniformly with the soil pressure beyond the distance, **d**, from the corbel, where **d** is the pad thickness. See Section 5-2.04C, *Horizontal Shear*, for additional information about horizontal shear.

8-2.05A(1) Uniform Corbel Spacing (Symmetrical Analysis)

The horizontal shear in a continuous pad with uniformly spaced posts is determined by:

$$\mathbf{f_v} = \left(\frac{3}{2}\right)\frac{\left\{\frac{1000P\left[\frac{L_b}{2}-\frac{\frac{m}{2}}{12}-\frac{\frac{t}{2}}{12}-\frac{d}{12}\right]}{L_b}\right\}}{bd}. \qquad \text{(8-2.05A(1)-1)}$$

where $\mathbf{f_v}$ = Horizontal shear stress in the pad on the long side (psi)

P = Post load (kips)

$\mathbf{L_b}$ = Total bearing length of the pad (ft)

m = Corbel spacing (ft)

t = Width of corbel (in)

d = Thickness of pad (in)

b = Width of pad (in)

8-2.05A(2) Non-Uniform Corbel Spacing (Asymmetrical Analysis)

The horizontal shear on the long side of the post in a continuous pad with non-uniformly spaced posts is determined by:

$$\mathbf{f_v} = \left(\frac{3}{2}\right)\frac{\left\{\frac{1000P\left[L_2-\frac{\frac{t}{2}}{12}-\frac{d}{12}\right]}{L_b}\right\}}{bd}. \qquad \text{(8-2.05A(2)-1)}$$

where $\mathbf{f_v}$ = Horizontal shear stress in the pad on the long side (psi)

P = Post load (kips)

$\mathbf{L_2}$ = Pad length on long side (ft)

t = Width of corbel (in)

d = Thickness of pad (in)

$\mathbf{L_b}$ = Total bearing length of the pad (ft)

b = Width of pad (in)

8-2.05B Pad Analysis at Interior Post with Uniform Post Spacing

Figure 8-8, *Pad at Interior Post with Uniform Spacing and Double Corbels*, shows a falsework bent where the post spacing (PS) is uniform along a continuous pad and the post load is distributed across the pad by double corbels.

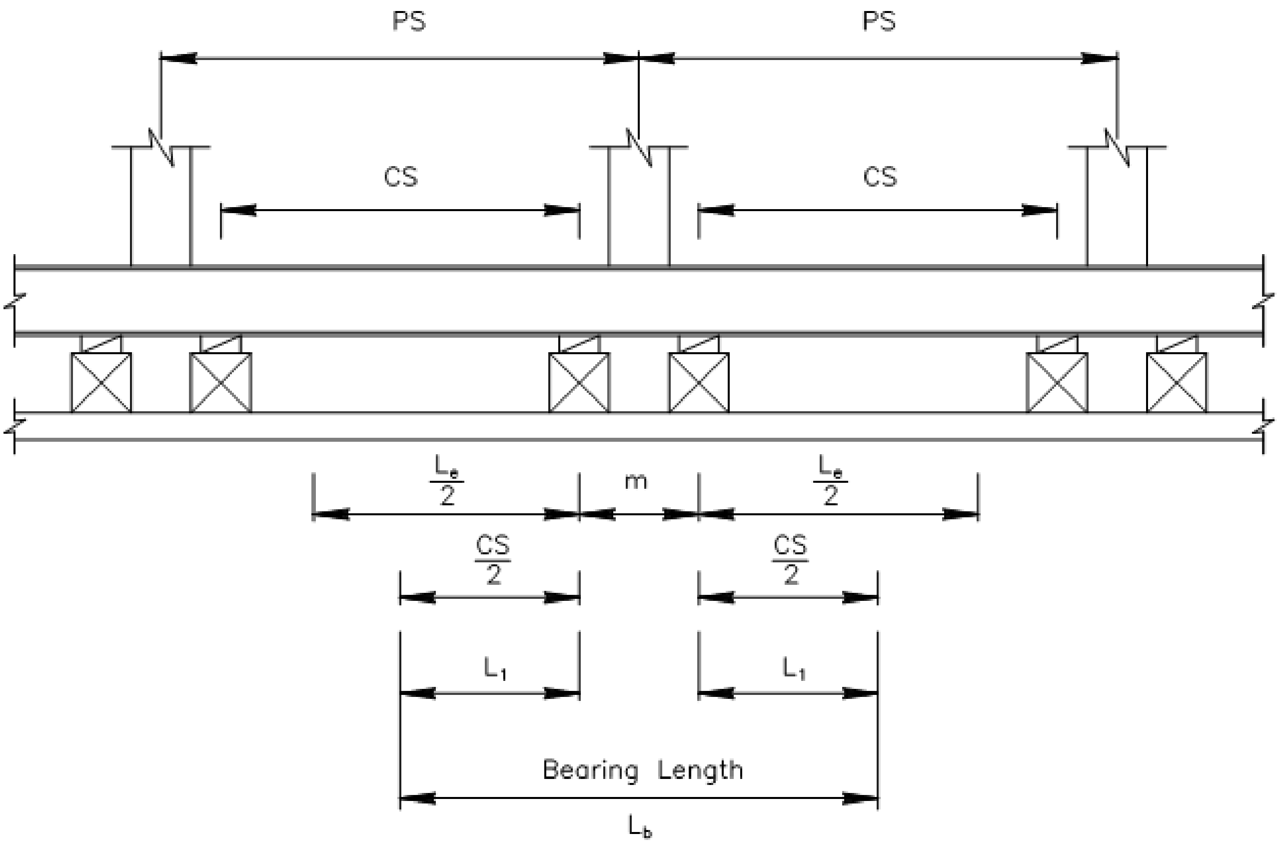


Figure 8-8. Pad at Interior Post with Uniform Spacing and Double Corbels.

When the post spacing is uniform, the bearing length is symmetrical. Analyze the pad as follows:

1. Calculate the effective length, $\mathbf{L_e}$, of the pad using the SYM formula. For this calculation, use the post load, **P**, not the load applied by the corbel. The pad responds to the post load by a system of two closely spaced corbels as though the load was applied by a single corbel:

$$\mathbf{L_e = L_{SYM}} \text{ from equation 8-2.02A-2} \qquad \text{(8-2.05B-1)}$$

2. The limiting length on each side, $\mathbf{L_1}$, is determined by:

$$\mathbf{L_1} = \textbf{ smaller of } \begin{cases} \frac{CS}{2} \\ \frac{\mathbf{L_e}}{2} \end{cases} \quad \text{(8-2.05B-2)}$$

3. The limiting bearing length, $\mathbf{L_b}$, is the sum of two times limiting lengths found in step 2 plus the corbel spacing, **m**:

$$\mathbf{L_b = 2L_1 + m} \quad \text{(8-2.05B-3)}$$

4. Calculate the soil pressure, $\boldsymbol{\sigma_b}$, using the limiting bearing length, $\mathbf{L_b}$, from step 3 and compare to the allowable soil bearing value. Use the equation 8-2.03-1 for $\boldsymbol{\sigma_b}$.

5. Calculate the pad stress, $\mathbf{f_v}$, due to horizontal shear using the limiting bearing length, $\mathbf{L_b}$, from step 3. Calculate the stress at a distance, **d**, from the face of the corbel where **d** is the pad thickness. Use equation 8-2.05A(1)-1 for $\mathbf{f_v}$.

8-2.05C Pad Analysis at Interior Post with Non-Uniform Post Spacing

Figure 8-9, *Pad at Interior Post with Non-Uniform Spacing and Double Corbels*, shows a falsework bent where the post spacing is non-uniform along a continuous pad and the post load is distributed across the pad by double corbels, When the post spacing is not uniform, the contribution to system adequacy made by the pad on one side of a post must be determined independently of the contribution made by the pad on the opposite side.

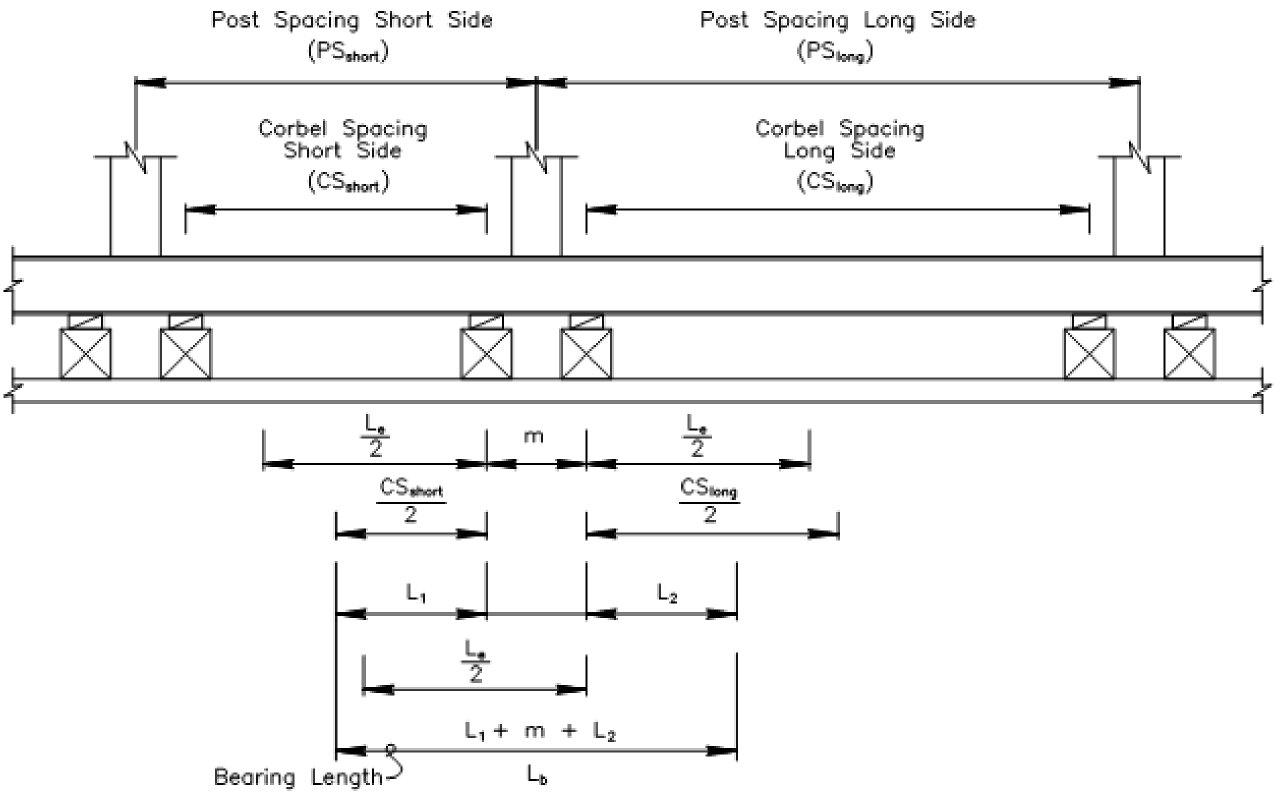


Figure 8-9. Pad at Interior Post with Non-Uniform Spacing and Double Corbels.

When the post spacing is non-uniform, the bearing length can be asymmetrical or symmetrical. Begin with the side that has the shorter post spacing. In this case the left side. Analyze the pad as follows:

1. Calculate the effective length, $\mathbf{L_e}$, of the pad using the SYM formula. For this calculation, use the post load, **P**, not the load applied by the corbel. The pad responds to the post load by a system of two closely spaced corbels as though the load was applied by a single corbel:

$$\mathbf{L_e = L_{SYM}} \text{ from equation 8-2.02A-2} \tag{8-2.05C-1}$$

2. The limiting length on the short side, $\mathbf{L_1}$, is determined by:

$$\mathbf{L_1 = \text{ smaller of }} \begin{cases} \dfrac{\boldsymbol{CS_{short}}}{\mathbf{2}} \\ \dfrac{\mathbf{L_e}}{\mathbf{2}} \end{cases} \tag{8-2.05C-2}$$

3. The limiting length on the long side, $\mathbf{L_2}$, is determined by:

$$\mathbf{L_2 = \ smaller\ of} \begin{cases} \dfrac{CS_{long}}{2} \\ \dfrac{L_e}{2} \end{cases} \tag{8-2.05C-3}$$

4. Asymmetrical Analysis: If $\mathbf{L_1 \neq L_2}$, the bearing length is asymmetrical. (If the lengths are equal, skip to step 5).

    a. The limiting bearing length, $\mathbf{L_b}$, is the sum of the limiting lengths found in step 2 and step 3c plus the corbel spacing (m):

    $$\mathbf{L_b = L_1 + m + L_2} \tag{8-2.05C-4}$$

    b. Calculate the soil pressure, $\boldsymbol{\sigma_b}$, using the limiting bearing length, $\mathbf{L_b}$, from step 4a and compare to the allowable soil bearing value. Use the equation 8-2.03-1 for $\boldsymbol{\sigma_b}$.

    c. Calculate the pad stress, $\mathbf{f_v}$, on the long side due to horizontal shear using the lengths $\mathbf{L_2}$ from step 3 and $\mathbf{L_b}$ from step 4a. Calculate the stress at a distance, **d**, from the face of the corbel where **d** is the pad thickness. Use equation 8-2.05A(2)-1 for $\mathbf{f_v}$.

5. Symmetrical Analysis: If $\mathbf{L_1 = L_2}$ the bearing length is symmetrical.

    a. The limiting bearing length, $\mathbf{L_b}$, is the sum of the length from Step 1 plus the corbel spacing, **m**:

    $$\mathbf{L_b = L_e + m} \tag{8-2.05C-5}$$

    b. Calculate the soil pressure, $\boldsymbol{\sigma_b}$, using the limiting bearing length, $\mathbf{L_b}$, from step 5a and compare to the allowable soil bearing value. Use the equation 8-2.03-1 for $\boldsymbol{\sigma_b}$.

    c. Calculate the pad stress, $\mathbf{f_v}$, due to horizontal shear using the limiting bearing length, $\mathbf{L_b}$, from step 5a. Calculate the stress at a distance, **d**, from the face of the corbel where **d** is the pad thickness. Use the equation 8-2.05A(1)-1 for $\mathbf{f_v}$.

8-2.05D Pad Analysis at Exterior Post

This section shows the procedures for timber pads at exterior posts with double corbels. Figure 8-10, *Pad at Exterior Post with Double Corbels* shows a double corbel configuration at an exterior post on a continuous pad and the post load is distributed across the pad by two corbels. For exterior posts, the contribution to the system adequacy made by the length of pad on the outside of the post must be determined independently of the contribution made by the pad on the inside. For typical bent configurations and post spacing, the pad length on the inside of the post will be the long side. The procedure is also valid in any case where the long side length is on the outside.

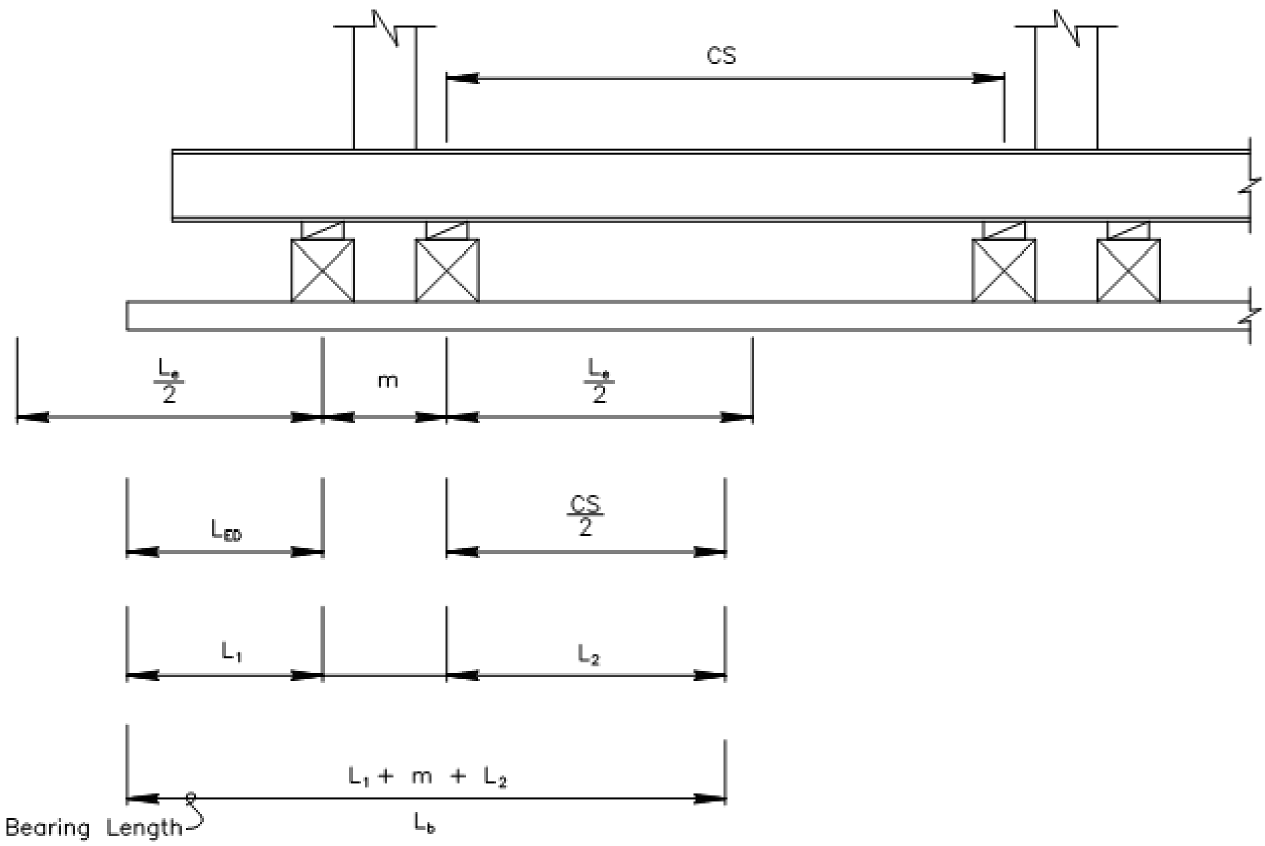


Figure 8-10. Pad at Exterior Post with Double Corbels.

For exterior posts the bearing length can be asymmetrical or symmetrical. Begin with the outside, in this case the left side. Analyze the pad as follows:

1. Calculate the effective length, $\mathbf{L_e}$, of the pad using the SYM formula:

$$\mathbf{L_e} = \mathbf{L_{SYM}} \text{ from equation 8-2.02A-2} \qquad \text{(8-2.05D-1)}$$

2. The limiting bearing length on the outside, $\mathbf{L_1}$, is determined by:

$$\mathbf{L_1 = \text{smaller of} \begin{cases} L_{ED} \\ \frac{L_e}{2} \end{cases}} \tag{8-2.05D-2}$$

3. The limiting length on the inside, $\mathbf{L_2}$, is determined by:

$$\mathbf{L_2 = \text{smaller of} \begin{cases} \frac{CS}{2} \\ \frac{L_e}{2} \end{cases}} \tag{8-2.05D-3}$$

4. Asymmetrical Analysis: If $\mathbf{L_1 \neq L_2}$ the bearing length is asymmetrical. (If the lengths are equal, skip to step 5):
   a. The bearing length, $\mathbf{L_b}$, is the sum of the limiting lengths found in step 3 and step 5c plus the corbel spacing, **m**:

$$\mathbf{L_b = L_1 + m + L_2} \tag{8-2.05D-4}$$

   b. Calculate the soil pressure, $\boldsymbol{\sigma_b}$, using the limiting bearing length, $\mathbf{L_b}$, from step 4a and compare to the allowable soil bearing value. Use equation 8-2.03-1 for $\boldsymbol{\sigma_b}$.
   c. Calculate the pad stress, $\mathbf{f_v}$, on the long side due to horizontal shear using the greater of $\mathbf{L_1}$ and $\mathbf{L_2}$ lengths from steps 2 and 3 and $\mathbf{L_b}$ from step 4a. Calculate the stress at a distance, **d**, from the face of the corbel where **d** is the pad thickness. Use equation 8-2.05A(2)-1 for $\mathbf{f_v}$.

5. Symmetrical Analysis: If $\mathbf{L_1 = L_2}$ the bearing length is symmetrical. This is unlikely to occur in actual practice:
   a. The limiting bearing length, $\mathbf{L_b}$, is the sum of the length from Step 1 plus the spacing, **m**:

$$\mathbf{L_b = L_1 + m + L_2} \tag{8-2.05D-5}$$

   b. Calculate the soil pressure, $\boldsymbol{\sigma_b}$, using the limiting bearing length, $\mathbf{L_b}$, from step 5a and compare to the allowable soil bearing value. Use equation 8-2.03-1 for $\boldsymbol{\sigma_b}$.
   c. Calculate the pad stress, $\mathbf{f_v}$, due to horizontal shear using the limiting bearing length, $\mathbf{L_b}$, from step 5a. Calculate the stress at a distance, **d**,

from the face of the corbel where **d** is the pad thickness. Use equation 8-2.05A(1)-1 for $\mathbf{f_v}$.

The same general procedure applies when the short side is on the inside of an exterior post.

8-2.06 Analysis of Individual Pads

The procedures for individual pads are similar to those used for continuous pads, as discussed in the following sections.

8-2.06A Analysis of Symmetrical Pads

Figure 8-11, *Symmetrical Individual Pads with Single Corbel,* shows two individual pads where the bearing length is symmetrical about the post centerline. Analyze the pad as follows:

1. Calculate the effective length, $\mathbf{L_e}$, of the pad using the SYM formula:

$$\mathbf{L_e = L_{SYM}} \text{ from equation 8-2.02A-2} \qquad (8\text{-}2.06A\text{-}1)$$

2. The limiting bearing length, $\mathbf{L_b}$, is determined by:

$$\mathbf{L_b = \text{smaller of} \begin{cases} L_{pad} \\ L_e \end{cases}} \qquad (8\text{-}2.06A\text{-}2)$$

3. Calculate the soil pressure, $\boldsymbol{\sigma_b}$, using the limiting bearing length, $\mathbf{L_b}$, from step 2 and compare to the allowable soil bearing value. Use equation 8-2.03-1 for $\boldsymbol{\sigma_b}$.
4. Calculate the pad stress, $\mathbf{f_v}$, due to horizontal shear using the length, $\mathbf{L_b}$, from step 2. Calculate the stress at a distance, **d**, from the face of the corbel where **d** is the pad thickness. Use equation 8-2.04A(1)-1 for $\mathbf{f_v}$.

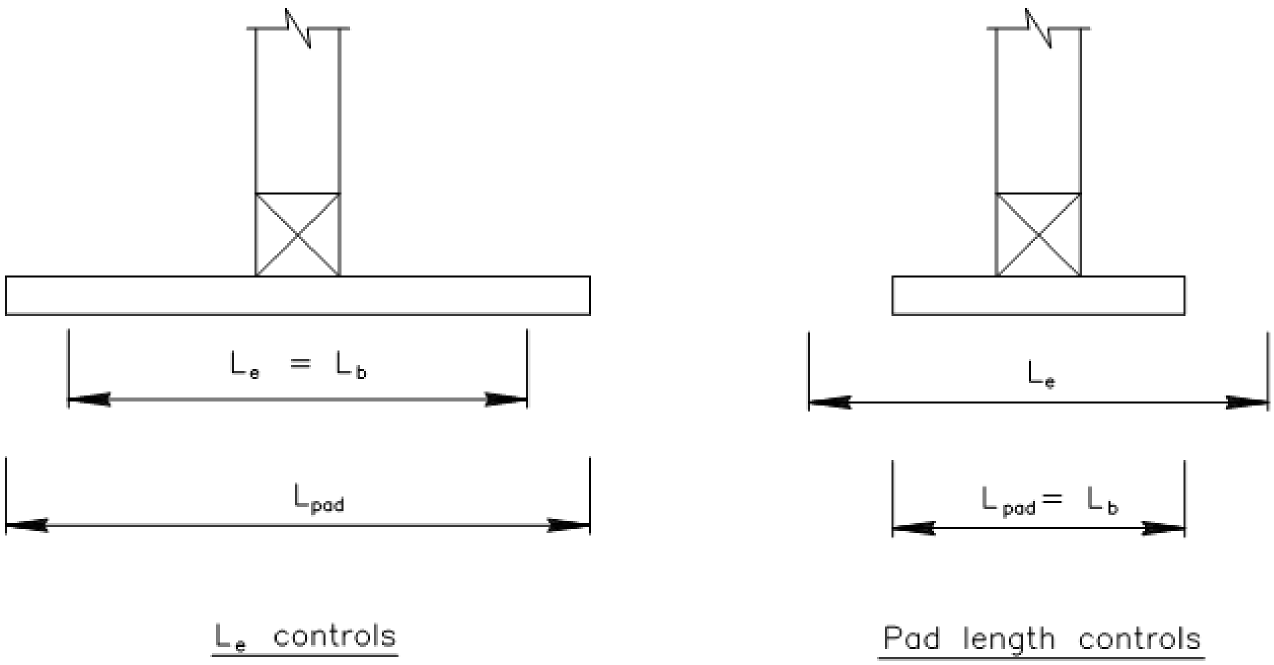


Figure 8-11. Symmetrical Individual Pads with Single Corbel.

8-2.06B Analysis of Asymmetrical Pads

Figure 8-12, *Asymmetrical Individual Pad with Single Corbel,* shows an individual pad where the bearing length is asymmetrical about the post centerline. Analyze the pad as follows:

1. Calculate the effective length, $\mathbf{L_e}$, of the pad using the SYM formula:

$$\mathbf{L_e = L_{SYM}} \text{ from equation 8-2.02A-2} \qquad (8\text{-}2.06B\text{-}1)$$

2. The limiting bearing length on the short side, $\mathbf{L_1}$, is determined by:

$$\mathbf{L_1 = \text{smaller of} \begin{cases} L_{ED1} \\ \frac{L_e}{2} \end{cases}} \qquad (8\text{-}2.06B\text{-}2)$$

3. The limiting bearing length on the long side, $\mathbf{L_2}$, is determined by:

$$\mathbf{L_2 = \text{smaller of} \begin{cases} L_{ED2} \\ \frac{L_e}{2} \end{cases}} \qquad (8\text{-}2.06B\text{-}3)$$

4. <u>Asymmetrical Analysis</u>: If $\mathbf{L_1 \neq L_2}$ the bearing length is asymmetrical. (If the lengths are equal, skip to step 5:)

a. The limiting bearing length, $\mathbf{L_b}$, is determined by:

$$\mathbf{L_b} = \mathbf{L_1} + \mathbf{L_2} \tag{8-2.06B-4}$$

b. Calculate the soil pressure, $\boldsymbol{\sigma_b}$, using the limiting bearing length, L_b, from step 5d and compare to the allowable soil bearing value. Use equation 8-2.03-1 for $\boldsymbol{\sigma_b}$.

c. Calculate the pad stress, $\mathbf{f_v}$, on the long side due to horizontal shear using the lengths $\mathbf{L_2}$ from step 3 and $\mathbf{L_b}$ from step 4a. Calculate the stress at a distance, **d**, from the face of the post or corbel where **d** is the pad thickness. Use equation 8-2.04A(2)-1 for $\mathbf{f_v}$. For some asymmetrical loading configurations, the limiting length on the long side, $\mathbf{L_2}$, will be <u>shorter</u> than the limiting length on the short side, $\mathbf{L_1}$, in which case the stress due to horizontal shear will be calculated on the short side.

5. <u>Symmetrical Analysis</u>: If $\mathbf{L_1 = L_2}$ the bearing length is symmetrical:

a. The limiting bearing length is determined by:

$$\mathbf{L_b} = \mathbf{L_1} + \mathbf{L_2} \tag{8-2.06B-5}$$

b. Calculate the soil pressure, $\boldsymbol{\sigma_b}$, using the limiting bearing length, $\mathbf{L_b}$, from step 5a and compare to the allowable soil bearing value. Use equation 8-2.03-1 for $\boldsymbol{\sigma_b}$.

c. Calculate the pad stress, $\mathbf{f_v}$, due to horizontal shear using the limiting bearing length, $\mathbf{L_b}$, from step 5a. Calculate the stress at a distance, **d**, from the face of the post or corbel where **d** is the pad thickness. Use equation 8-2.04A(1)-1 for $\mathbf{f_v}$.

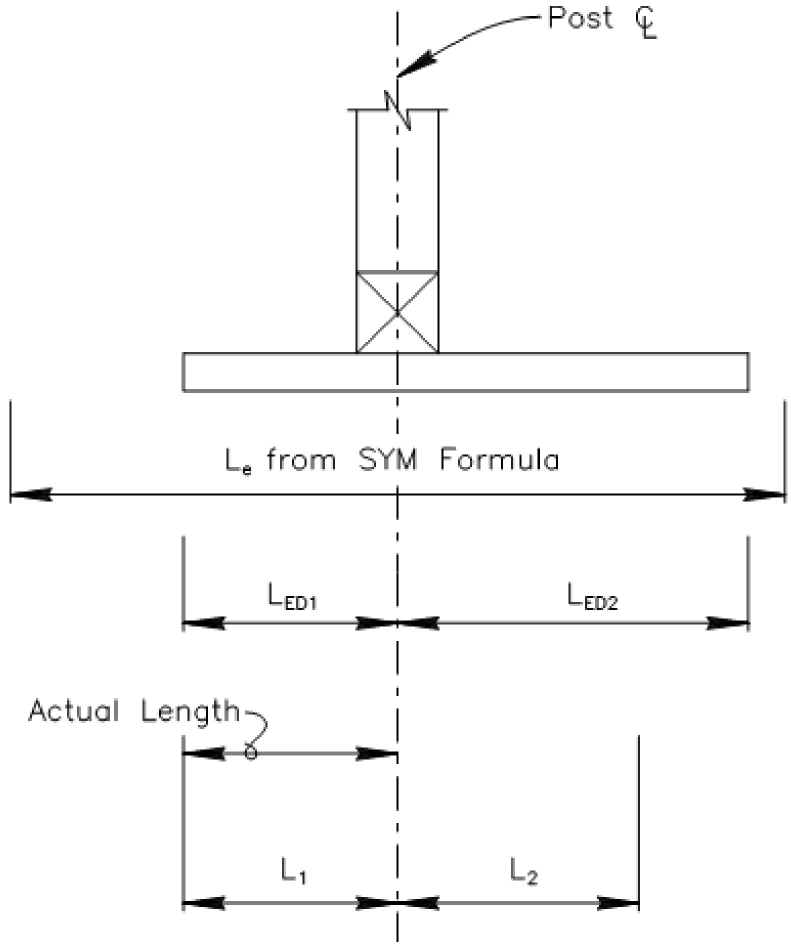


Figure 8-12. Asymmetric Individual Pad with Single Corbel.

8-2.07 Full Width Joints in Continuous Pads

Full width joints (points of pad discontinuity) in continuous pad systems are an important design consideration for continuous pads. The discontinuity at the joints directly affects the ability of a continuous pad to distribute the post load uniformly. The joint locations must be planned in advance and shown on the shop drawings.

Supplemental pads or doubler pads should be provided at joints in continuous pads to achieve uniform load distribution as assumed in the analysis. The supplemental pads and doubler pads must be shown on the shop drawings.

If joints are anticipated in the design, without supplemental pads or doubler pads, the pad must be analyzed as follows:

- A section with only one post between the joint and the end of the pad must be analyzed as an individual pad.

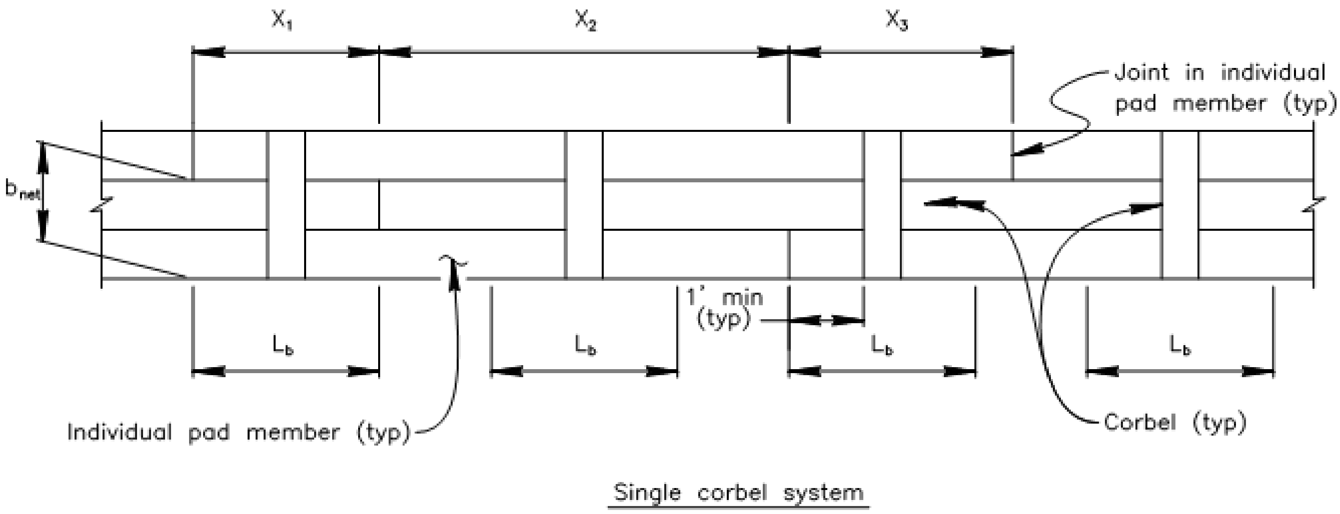
- A section with two or more posts between the joint and the end of the pad must be analyzed as continuous where both the post at the end and the post at the joint are considered exterior posts.
- A section with only one post between two joints must be analyzed as an individual pad.
- A section with two or more posts between two joints must be analyzed as continuous where the posts nearest each joint are considered exterior posts.

8-2.08 Joints in Individual Pad Members

Joints in individual members of continuous pads must be located outside the effective bearing length or at the midpoint between adjacent posts or corbels.

8-2.08A Supplemental Pads

To facilitate construction, some contractors intentionally over design a continuous pad system by providing a greater overall pad width, and a correspondingly greater number of adjacent pad members, than required by theoretical design considerations. The supplemental pads provide redundancy in the pad system.



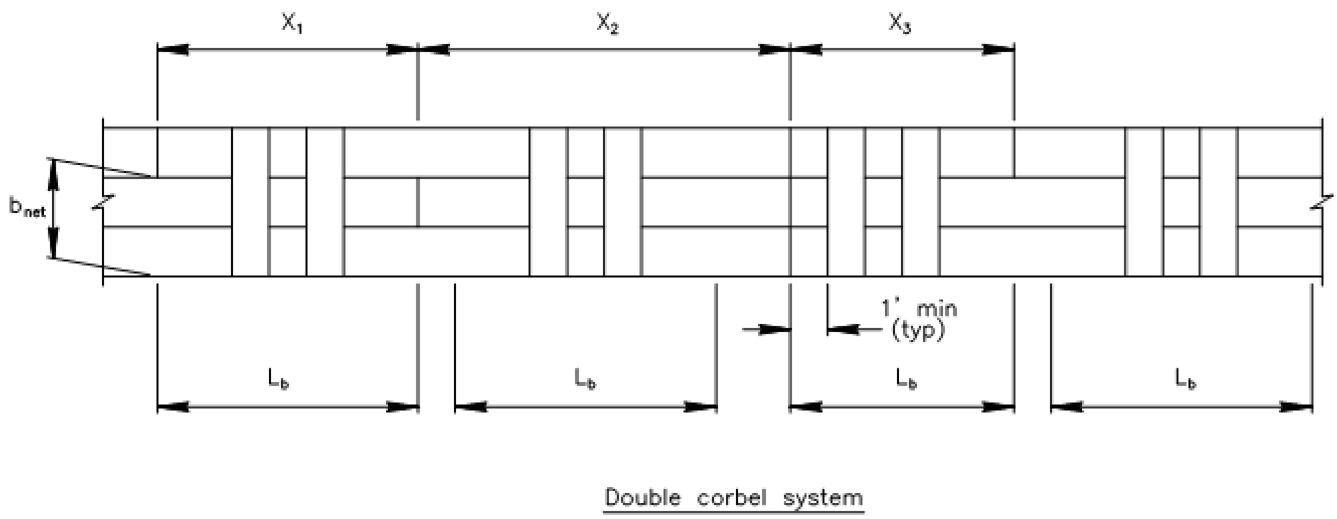


Figure 8-13. Supplemental Pads.

Referring to Figure 8-13, *Supplemental Pads*, when supplemental pads are provided, joints in individual members of the continuous pad system are acceptable, subject to the following restrictions:

- Joints in individual pad members of continuous pads must be staggered a minimum distance, **x**:

$$\mathbf{x_i} \geq \begin{cases} \mathbf{4\ ft} \\ \mathbf{L_b} \end{cases} \tag{8-2.08A-1}$$

where $\mathbf{x_i}$ = Distance between joints in individual pad members

L_b = Limiting bearing length required by the nearest post or corbel as defined in Sections 8-2.04, *Continuous Pad with Single Corbel,* and 8-2.05, *Continuous Pad with Two or More Corbels.*

- Joints in individual pad members of continuous pads are not allowed under single corbels or multiple corbel systems and must be located a minimum of 1-foot away from single corbels and multiple corbel systems.
- At any given joint location of an individual member, the net width of the continuous pad system, $\mathbf{b_{net}}$, may not be less than the pad width, **b**, required if supplemental pads were not used:

$$\mathbf{b_{net}} \geq \mathbf{b} \qquad \text{(8-2.08A-2)}$$

where $\mathbf{b_{net}}$ = The pad width remaining after deducting the width of all individual pad members having joints at the location under consideration

b = Pad width required if supplemental pads were not used

Since supplemental pads are not considered in the analysis, they must be clearly identified as such on the shop drawings.

8-2.08B Doubler Pads

A doubler pad is a second pad placed on top of the main pad. Doubler pads may be used to carry the post load across a joint in the main pad.

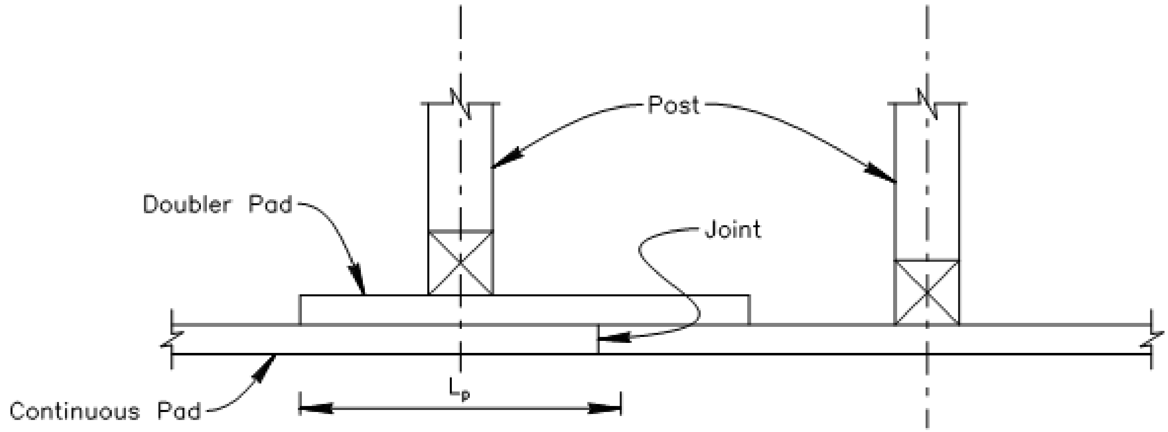
Refer to Figure 8-14, *Doubler Pads*, the phantom length, $\mathbf{L_P}$, is the adjusted effective length, $\mathbf{L_{ae}}$, of a symmetrically loaded "phantom" pad designed in accordance with Section 8-2.06, *Analysis of Individual Pads*.

To maintain the integrity of a continuous pad system, doubler pads must conform to the criteria below:

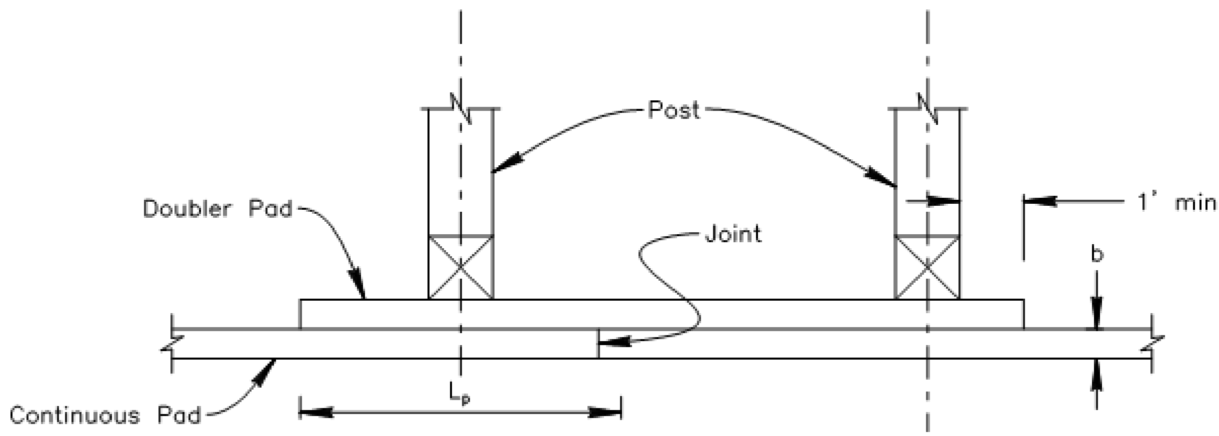
- Placed as an individual pad at a given post location or as a continuous pad between two or more posts.
- Be of the same width and thickness as the main pad.
- If a joint in the main continuous pad falls within the zone established by the phantom length, $\mathbf{L_P}$, for that post, install doubler pads using one of the following options:

- Install an individual doubler pad as shown in Figure 8-14, *Doubler Pads, Case A*. The pad must cover the entire phantom length zone, $\mathbf{L_P}$, and extend a minimum distance of 2 feet past the joint away from the post.
- Install a continuous doubler pad as shown in Figure 8-14, *Doubler Pads, Case B*. The pad must cover the entire phantom length zone, $\mathbf{L_P}$, and extend a minimum distance of 1 foot past the adjacent post.

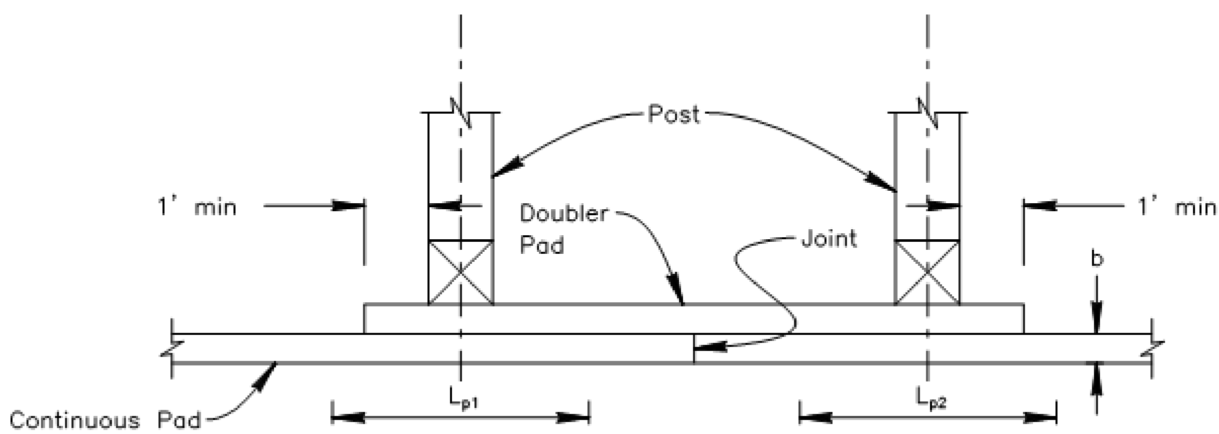
- If a joint in the main continuous pad falls outside the zones established by the phantom lengths, $\mathbf{L_{p1}}$ and $\mathbf{L_{p2}}$ for the adjacent posts, install the doubler pad as follows:
  - Install a continuous doubler pad as shown in Figure 8-14, *Doubler Pads, Case C*. The pad must extend a minimum distance of 1 foot past both adjacent posts.



Case A



Case B



Case C

Figure 8-14. Doubler Pads.

8-3 Concrete Pads

Concrete pads may be used as an alternative to timber pads. See Section 7-6, *Concrete Pads,* for concrete pads that are authorized for use on projects in California and for design requirements for other concrete pads that have not been authorized.

8-4 Soil Load Tests and Soil Bearing Values

8-4.01 Introduction

In the case of bridge foundation design, determining the supporting capacity of a given foundation material with sufficient accuracy to ensure an adequate structural design requires a complete foundation investigation by an experienced and capable engineering geologist or soils engineer. Fortunately, the sophisticated approach to foundation design, which is required for permanent work, is generally unnecessary for falsework, because in most falsework designs maximum bearing pressure is applied for only a short period of time and relatively greater settlements may be tolerated.

The *Standard Specifications*, Section 48-2.03B, *Temporary Structures – Falsework – Construction - Foundations,* include a provision which requires the contractor to demonstrate by suitable load tests that the soil bearing values assumed in the falsework design do not exceed the supporting capacity of the soil. This requirement is included in the specifications to further verify the adequacy of the falsework foundation, and the engineer should not hesitate to order a soil load test if he has doubt as to the ability of the foundation material to support the falsework loads. However, soil bearing capacity may in most cases be determined with sufficient accuracy for falsework design purposes by simple static load tests performed by the contractor. Ordinarily, it will not be necessary to employ the services of a private soil laboratory or consultant.

The following information has been prepared to assist the structure representative in those situations where a load test is necessary to verify assumed soil bearing values.

8-4.02 General Information

Soil load tests should be made at the location where falsework will be erected. Bearing pads for the test load must be set on the same material as the falsework footing, and soil moisture content must closely approximate the content expected during falsework use. If the soil moisture content changes due to rain, for example, it is necessary to retest the soil to determine if the soil bearing value has decreased.

The larger the bearing area of the test load pad, the more reliable the results. Pad area should be not less than 2 square feet. For silty or clayey materials, a minimum test load pad area of 3 square feet is preferred.

A load test made on a relatively weak soil, such as clay or silt, will satisfactorily demonstrate the bearing capacity of the surface strata. Greater care should be exercised for tests where small footings are used as these are more critical than larger footings in this type of soil.

A load test made on a thick layer of granular soil overlying a thin weak soil will demonstrate the capacity of the upper layer. It will tell little of the capacity of the lower layer since the test load is small and the pressure on the lower area may be almost negligible since it is spread over a large area.

The effect of a unit load on a small area may not correspond to the effect of the same unit load on a large area. A load of short duration on a plastic soil may not have the same effect as the same unit load on a large area of longer duration. However, this is not true for firm granular soils, as load duration does not affect this type of soil.

8-4.03 Load Test Procedure

As provided in the *Standard Specifications*, Section 48-2.03B, *Temporary Structures – Falsework – Construction – Foundations*, the contractor is responsible to perform load tests when requested by the structure representative. However, the structure representative must determine the suitability of the proposed test for the given site conditions and evaluate the test results. Division of Engineering Services (DES) Geotechnical Services is available for consultation and advice as to the suitability of load tests in a given field situation, as well as interpretation of test results.

To achieve uniformity, a load test as this term is used in the specifications means a test in which both settlement and duration of load are considered.

One simple and satisfactory test method is to apply a gradually increasing load with respect to a fixed time interval, and to record the settlement at the end of each time period. The soil yield point is reached when a small increase in load produces a large increase in settlement. The load at yield point should be divided by **FS = 2** to determine the allowable bearing value.

Table 8-1, *Sample Load Test,* and Figure 8-15, *Sample Load Test – Load vs. Settlement*, present the results of an example load test where the load was increased every 12 hours over a three-day period.

Table 8-1. Sample Load Test.

Time Interval	Total Time	Load	Settlement
(hours)	(hours)	(ksf)	(in)
12	12	2.0	0.2
12	24	4.0	0.6
12	36	5.0	1.2
12	48	6.0	2.0
12	60	6.5	2.8
4	64	7.0	4+

Load test results should be plotted as shown Figure 8-15, *Sample Load Test – Load vs. Settlement*.

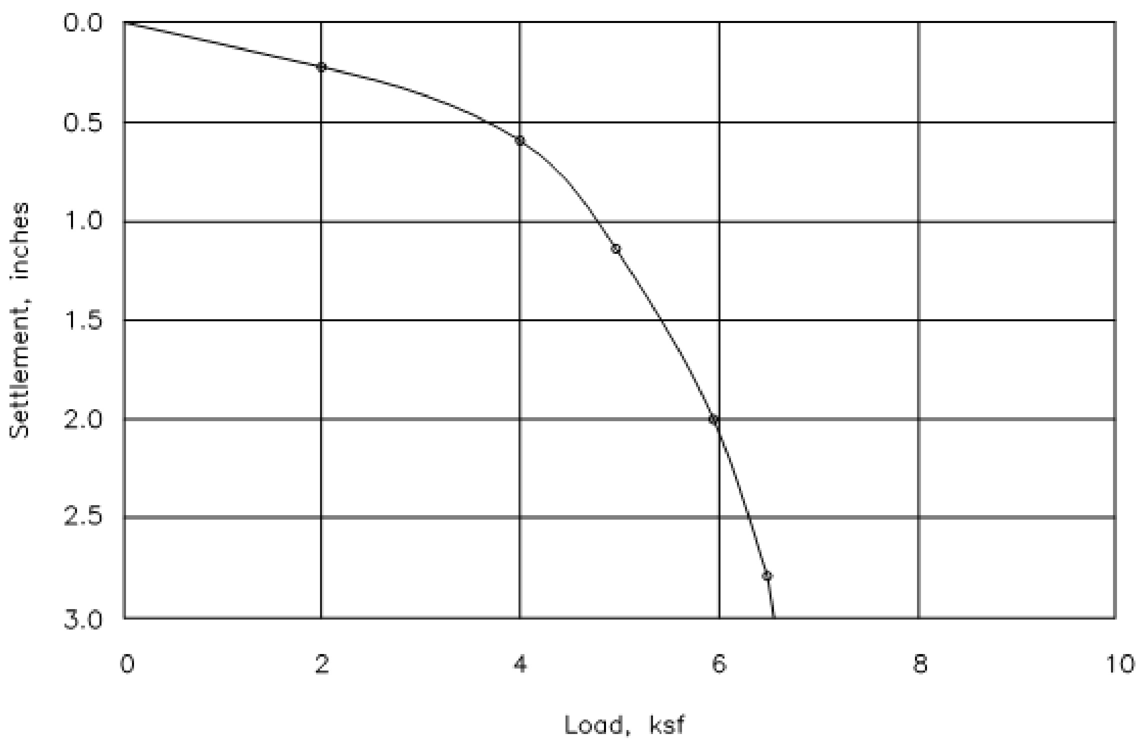


Figure 8-15. Sample Load Test – Load vs. Settlement.

Figure 8-15, *Sample Load Test – Load vs. Settlement,* shows that the soil yield point is about 6 (ksf) for this test, because the increase in the load from 4 to 6 ksf results in a large increase in the settlement. This value should be divided by **FS = 2** to determine the soil bearing value at the ground surface, which in this case is about 3 ksf.

If no clearly defined yield point exists, as will be the case in granular materials, the load which produces a 1-inch settlement may be taken as the ultimate bearing capacity. Again, this value should be divided by **FS = 2** to determine the allowable bearing value.

Referring to Figure 8-16, *Simple Static Soil Load Test Using K-Rail*, sections of K-rail are used as test load on four 1 foot wide by 2 feet long timber pads. The settlement is monitored with an optical level. The load will be incrementally increased until the settlement is about 1-inch, or the yield point is achieved. The yield point is achieved when a small increase in load produces a large increase in settlement.

Figure 8-16. Simple Static Soil Load Test Using K-Rail.

Another method takes into consideration the ratio of the size of the test pad to the size of the proposed pad, along with the contractor's anticipated settlement. In this method the general equation for determining the total load which may be supported by a given soil is expanded to include perimeter shear, as shown by the following relationship:

$$\mathbf{W = Ap = An + Pm} \qquad (8\text{-}4.03\text{-}1)$$

where **W** = Total load (lb)

A = Pad area (ft^2)

p = Allowable soil bearing value in (psf)

n = Compressive stress on the soil column directly beneath the pad (psf)

P = Pad perimeter (ft)

m = Perimeter shear in (plf)

If the ratio of pad perimeter to the pad area is **x**, where **P/A = x**, then the allowable soil bearing stress is:

$$\mathbf{p} = \frac{W}{A} = \mathbf{mx} + \mathbf{n} \qquad \text{(8-4.03-2)}$$

Values of **m** and **n** are found by test loading two or more pads having different areas and perimeters. The load which produces the contractor's assumed pad settlement is taken as the allowable stress. See Appendix D, *Example Problems*, Example 23, *Soil Bearing Load Test*.

8-4.04 Underlying Weak Strata

Test results, as discussed thus far, give only an indication of the allowable soil bearing values at the surface. If a weak underlying stratum exists, as indicated in the log of test borings, consideration should be given as to whether this stratum will support the falsework load without excessive settlement.

An assumption can be made that the load is spread with the depth at a 1:2 (H:V) slope as shown in Figure 8-17, *Load Dispersion to Weakest Soil Strata*.

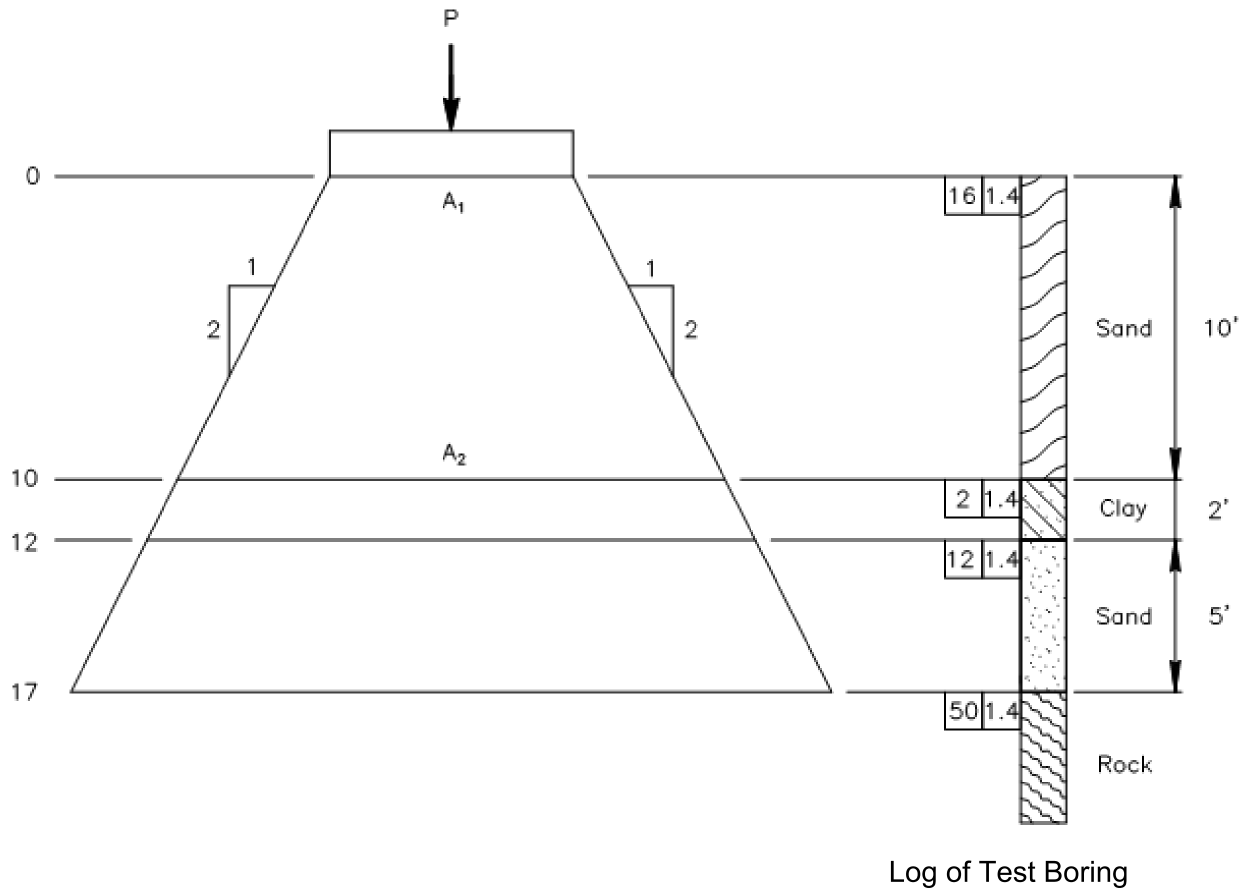


Figure 8-17. Load Dispersion to Weakest Soil Strata.

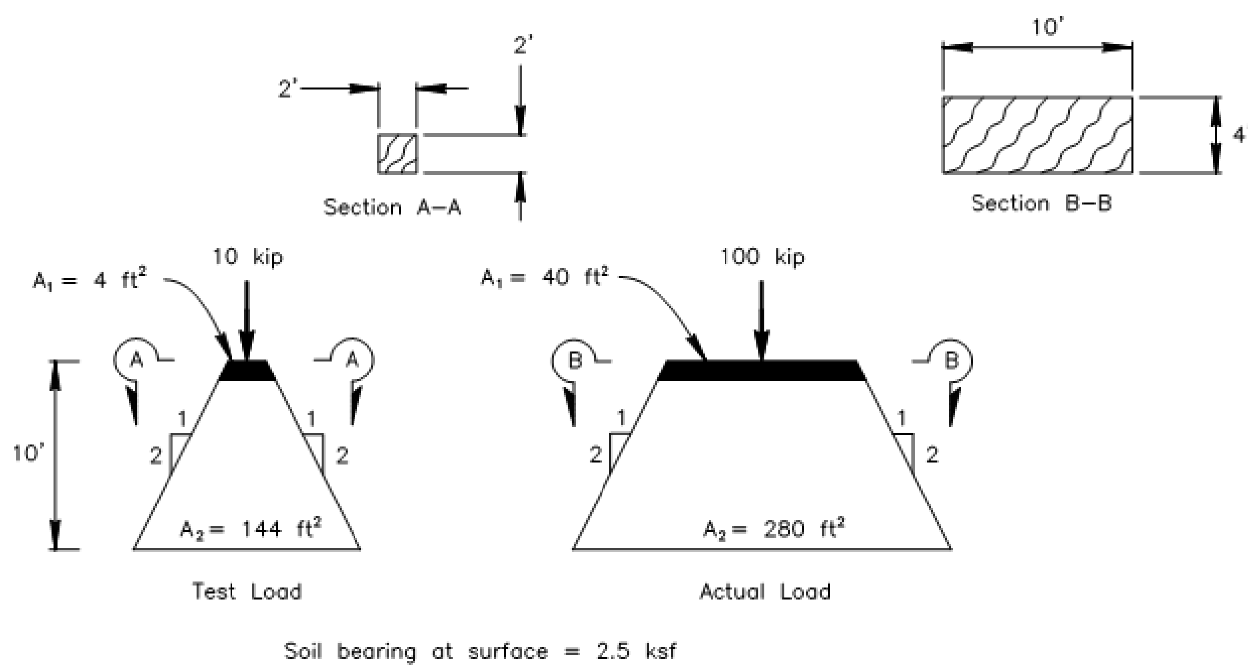


Figure 8-18. Simple Static Soil Load Test.

Figure 8-18, *Simple Static Soil Load Test,* shows a test load and an actual load spread onto the top of a weak strata 10-feet below. The figure provides the following information:

- The soil pressure at the surface is 2.5 ksf for both the test and the actual pad.
- The pressure on the weak underlying strata in the test load is 0.069 ksf, a reduction of 36:1 due to load spreading.
- In the actual condition the pressure is 0.36 ksf, a reduction of only 7:1, and this pressure may be more than the weak strata can safely support.

To assist with the analysis, charts showing allowable soil pressure for sandy and clayey soils are shown in Figure 8-19, *Allowable Bearing on Sandy Soils,* and Figure 8-20, *Allowable Bearing on Clayey Soils*, respectively. These charts give a general idea of the allowable bearing, based on soil classification.

8-4.05 Settlement

The anticipated settlement of falsework is limited to 1-inch by the *Standard Specifications*, Section 48-2.02B(1), *Materials – Design Criteria – General*. The engineer reviewing the shop drawings must be able to assess the probability that a given settlement, as predicted by the contractor, will actually occur. The general statements below may help in predicting these settlements:

- Granular Material: The maximum settlement will occur under the load as it is applied and is usually small in magnitude.
- Silt and Fine Sand: A large part of the settlement occurs as the load is applied. More occurs as the water is squeezed out under long-term loading. If the water table rises, a “quick” condition may result with floatation of the fine grains and a resulting settlement increase at this later date.
- Clay: Part of the consolidation occurs as the load is applied, but the rate of consolidation decreases with time. Settlement may also occur due to drying out of clay in the summer. All settlement is due to a squeezing out or loss of moisture in the clay.

8-4.06 Soil Bearing Values

In general, allowable soil bearing capacity in California varies from:

$$\mathbf{2000psf \leq p \leq 4000psf} \tag{8-4.06-1}$$

where **p** = Allowable soil bearing capacity

However, there are instances where the soils are very weak, e.g. bay mud, peat, and wetlands. In such instances, contractors may elect to use one of the following options:

- Design the pads for the low soil bearing capacity and settlement limitation.
- Implement remedial measures, such as soil stabilization or surcharging, to enhance soil bearing capacity.
- Design the falsework with pile foundation.

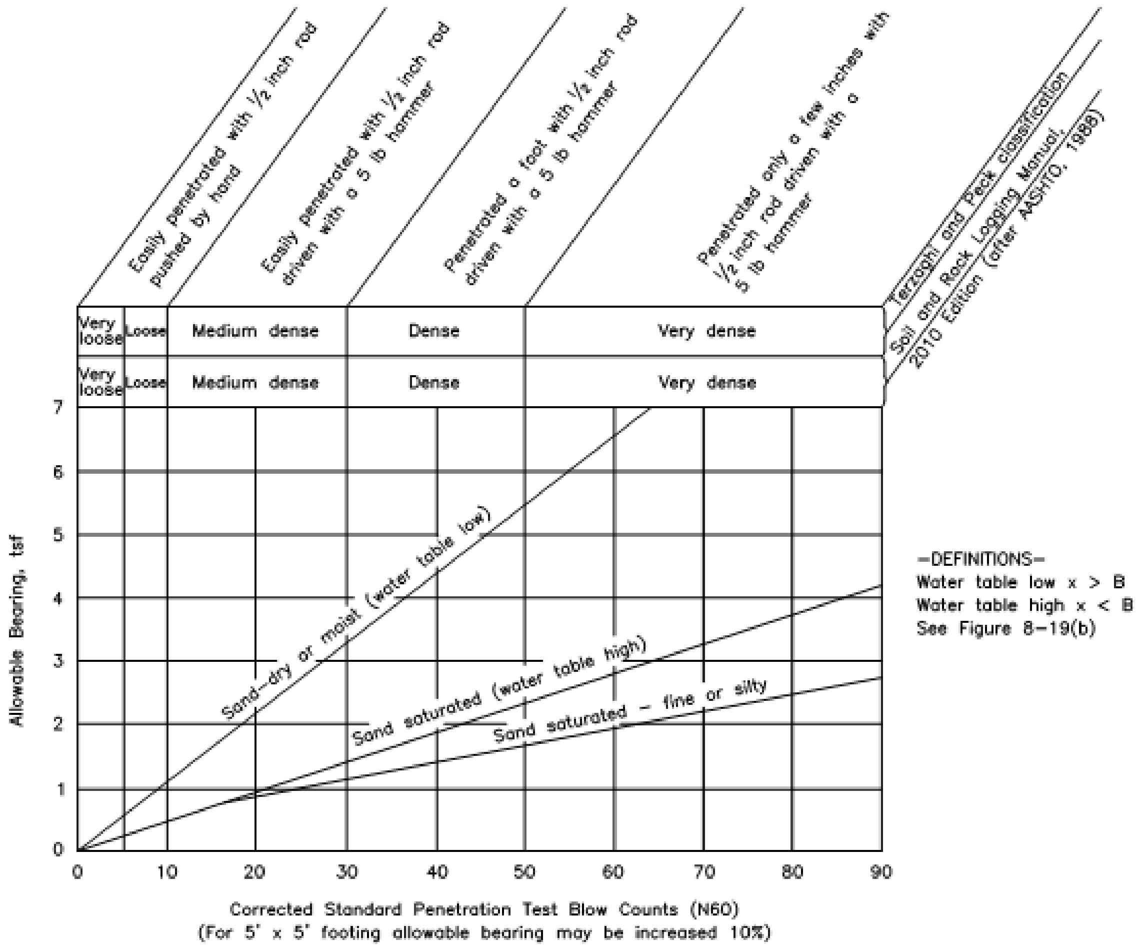
The engineer reviewing the shop drawings must use the available resources to verify that the contractor has used good engineering basis to assume an allowable soil bearing capacity for the falsework design. The available resources are for example:

- Log of test boring in the contract plans.
- Figure 8-19, *Allowable Bearing on Sandy Soils,* and Figure 8-20, *Allowable Bearing on Clayey Soils.*
- Geotechnical Services.

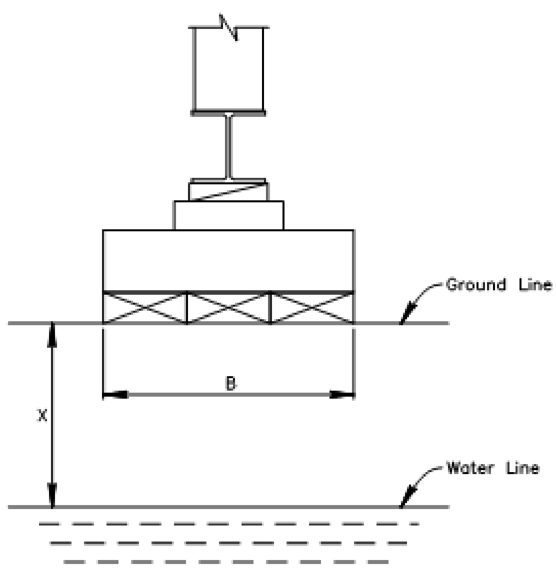
The structure representative should order a soil load test if there is any doubt as to the ability of the foundation material to support the falsework loads. Some reasons to request a load test are listed below, but are not limited to:

- Relatively high soil bearing capacity specified on the shop drawings.
- Poor compaction of the soil bearing the falsework load.
- Inconsistency in the type of import fill material placed.

Soil tests performed prior to a rainstorm may not be valid after the storm, for example if water puddles around the pads. The same is true if water puddles around the pads for any other reasons such as a water pipe break or curing water leakage.



(a) Allowable soil bearing versus corrected standard penetration test blow count



Notes for Figure 8–19:

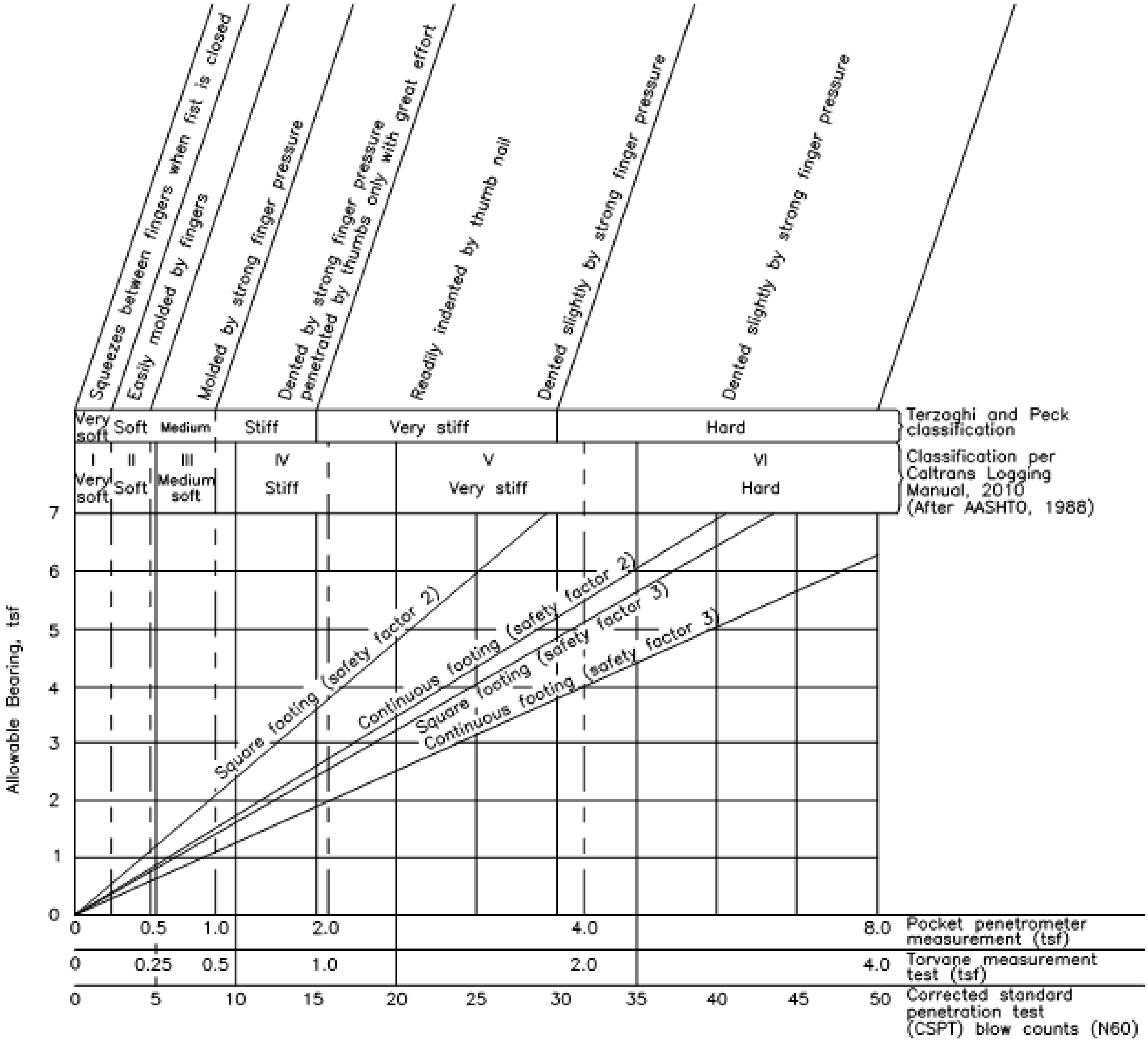
1. Based on factor of safety of 3.00 and settlement of <1 inch (footings 10' x 10')
2. It should not be used for footings on slopes
3. It was developed assuming uniform underlying soil

Major factors affecting bearing capacity:

1) Position of water table
2) Density of sand
3) Width of footing "B"

(b) Water table definition

Figure 8-19. Allowable Bearing on Sandy Soils.



Bearings as given above will generally be conservative for clayey soils

Notes:

1. Weak strata at some distance below footings may, in some cases, cause more settlement than soil layers immediately below the footings
2. For the same unit pressure, large footings settle most. This is particularly so where clay strata are involved
3. Greatest settlement may gnerally be expected at centers of loaded areas
4. Shear failures are most apt to occur when:
   1) Footings are small
   2) Settlements are large

Settlements tend to increase with the following:

1) Softness of the clayey material
2) Thickness of the compressible strata
3) Closeness of clay stratum to ground surface
4) Amount proposed loading exceeds past loading
5) Width of footing or loaded area
6) Height of water table
7) Liquid limit

Figure 8-20. Allowable Bearing on Clayey Soils.

8-5 Corbels

8-5.01 Introduction

Corbels are short beams used to distribute the post load or lower cap load across the top of pads.

Corbels must extend across the full width of the pad even though extension of the corbel to the outside of the pad may not be needed by theoretical design considerations, but it prevents cross grain bending in the pads.

The procedure for analyzing corbels is based on the following assumptions:

- The post load is applied symmetrically to the corbel and is uniformly distributed across the full width of the pad. The assumed symmetry may not be valid in the case of a continuous pad system where one or more supplemental pads are used to facilitate joint location. However, assuming a symmetrical load distribution gives conservative results when supplemental pads are used, and the assumption greatly simplifies the calculations.
- The corbel acts as a cantilever beam when resisting the load applied by the pad.
- For timber corbels, the point of fixity of the cantilever beam and the point of maximum bending moment is located mid-way between the centerline and outside face of the post, i.e. the 1/4 point of the post.
- For steel beam corbels, the point of fixity of the cantilever beam and the point of maximum bending moment is located at outside face of the post.
- If a round post is used, the post width to be used in the analysis is the length of the side of an equivalent area square post.

8-5.02 Timber Corbels

Figure 8-21, *Timber Corbel Flexure and Shear Dimensions,* shows a typical timber corbel system where the post is rectangular. Analyze the corbel as follows:

- Calculate the perpendicular-to-grain bearing stress of the corbel at the interface between post and corbel. If the applied stress exceeds the allowable stress, the system must be redesigned. The following redesign options may be used:
    - Reduce the post load.
    - Distribute the post load over a larger bearing area using a properly sized steel plate. When a steel plate is used, the analysis assumes that the post width is numerically equal to the length of the steel plate.

- Calculate the shear at a distance from the face of the post equal to the depth of the corbel **d**. Use the length L_H for the length of the distributed soil load for the shear calculation. Calculate the horizontal shearing stress at this location, see Section 5-2.04C, *Horizontal Shear*, and compare to the allowable horizontal shearing stress.
- Calculate the bending moment and the bending stress, see Section 5-2.04B, *Bending and Deflection*, and compare to the allowable bending stress. Use the length L_f as the length of the cantilever.

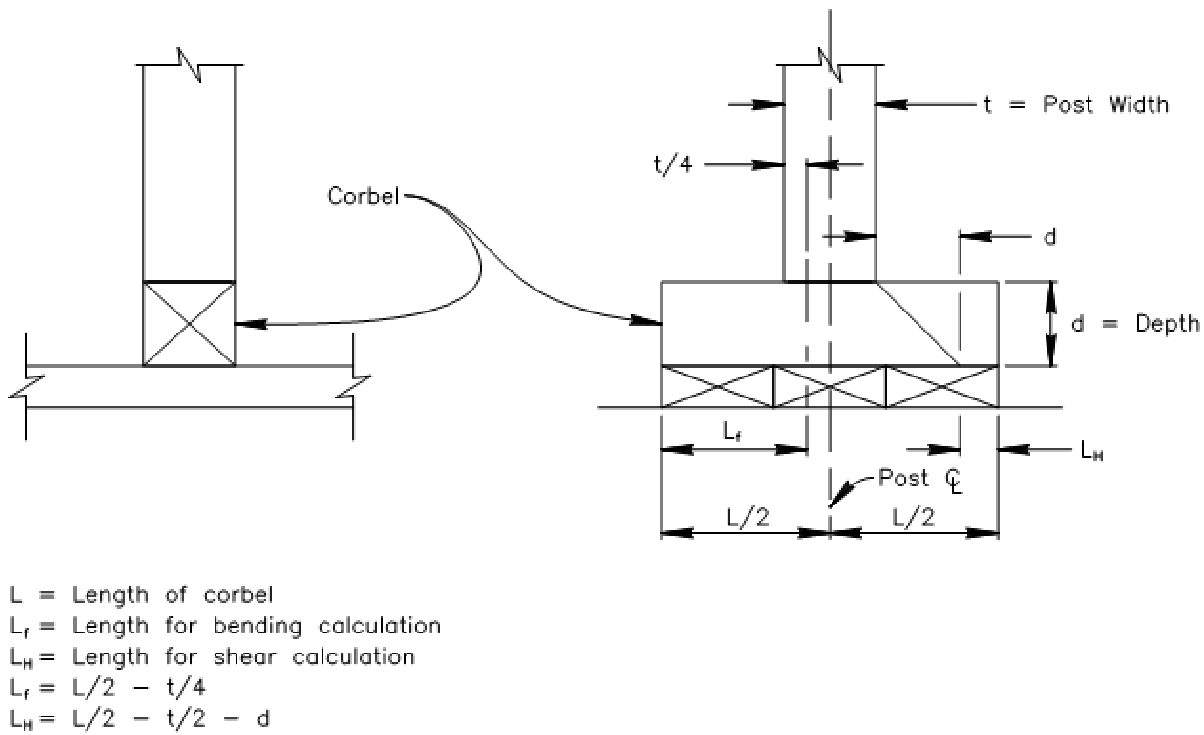


Figure 8-21. Timber Corbel Flexure and Shear Dimensions.

8-5.03 Steel Corbels

For steel beam corbels, analyze the corbel as follows:

- Calculate the web yielding and web crippling stress under the post using the total post load, see Sections 5-4.07, *Web Yielding,* and 5-4.08, *Web Crippling*. If the calculated stress exceeds the allowable stress, the system must be redesigned. The following redesign options may be used:
  - Reduce the post load
  - Increase the bearing length
  - Increase the web thickness

- Stiffen the web

- Calculate the shear stress on the beam web using 1/2 of the total post load.
- Calculate the bending moment and the bending stress. For steel beam corbels, the cantilever length is measured from the face of the post.
- Calculate the perpendicular-to-grain bearing stress in the pad at the interface between corbel and pad.

8-6 Pile Foundations

8-6.01 Introduction

In general, pile foundations will be required whenever site conditions preclude the use of timber or concrete pads. Typically, piles are used to support:

- Falsework over water.
- Falsework and heavy duty shoring where leg loads are high.
- Any type of falsework where differential settlement must be prevented.
- Any type of falsework where a conventional foundation is not feasible because of poor soil conditions.

Timber, steel, precast concrete, and cast-in-drilled-hole (CIDH) concrete piles may be used for falsework foundations. Steel piles are the most common type because they are easy to drive in most soil conditions, have a higher load carrying capacity than timber piles, and are more cost efficient than concrete piles. CIDH piles are rarely used due to the high associated cost. CIDH piles and precast concrete piles are not discussed in this manual.

Piles that are cut off and capped near the ground line will carry the superstructure load by braced falsework bents erected on top of the pile cap. In this configuration the piles are supported throughout their length and therefore are only subjected to axial loading. See Figure 8-22, *Piles Capped Close to Ground*.

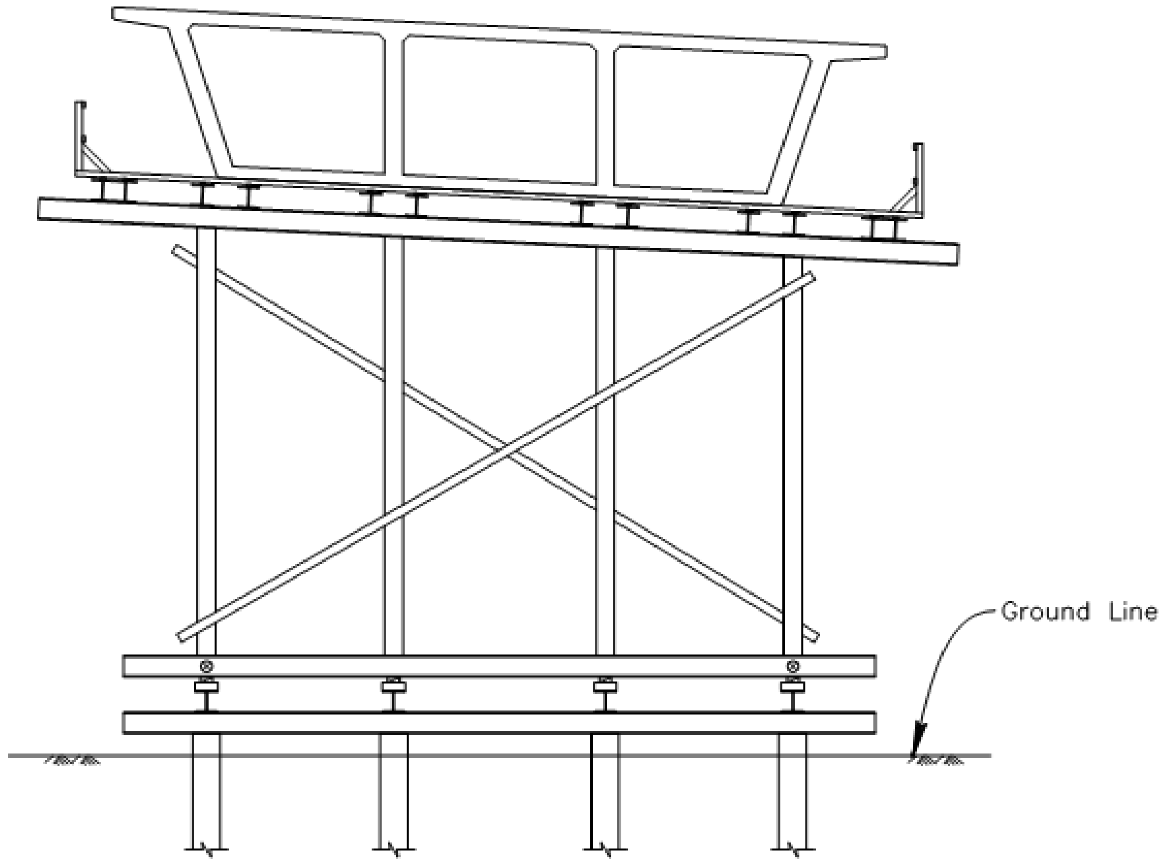


Figure 8-22. Piles Capped Close to Ground.

Site conditions may dictate the use of pile bents, where the piles extend above the ground surface. Such bents may be unbraced, partly braced or fully braced depending on site conditions, see Figure 8-23, *Pile Bent*.

Bracing systems are very effective in resisting horizontal forces in pile bents. When investigating the ability of the bracing to prevent collapse, the horizontal force produced by vertical load eccentricity (pile lean) must be considered. When investigating overturning stability, any theoretical uplift resistance provided by the piles must be neglected.

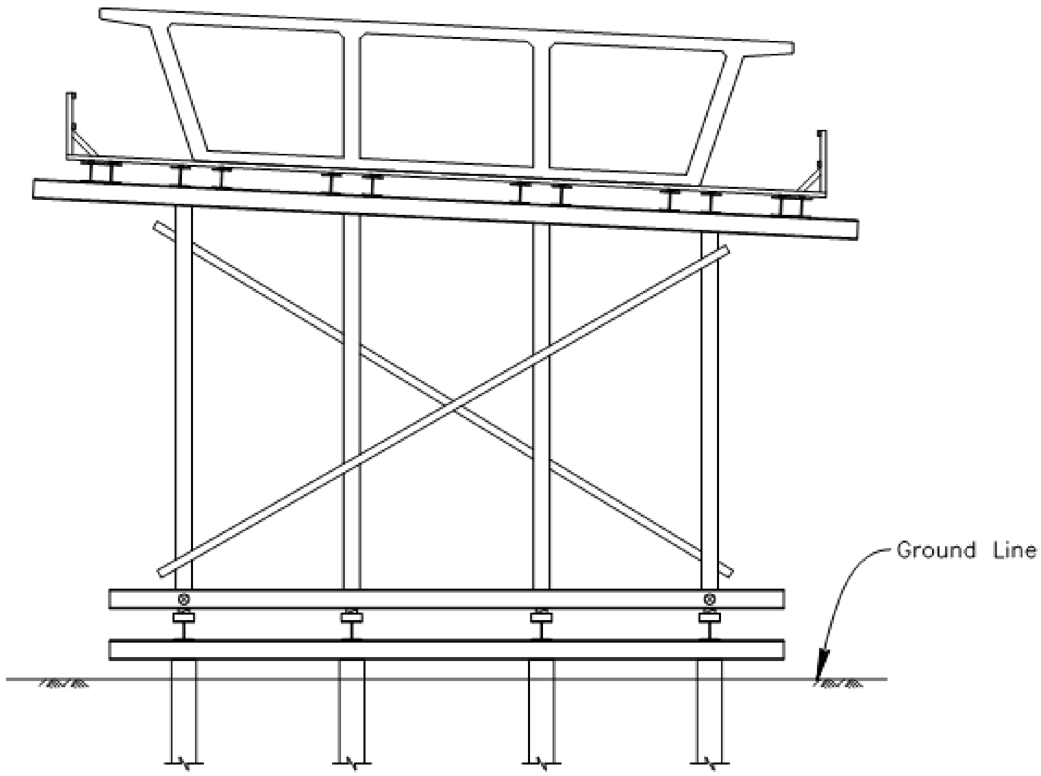


Figure 8-23. Pile Bent.

8-6.02 Pile Resistance

Piles that are plumb and properly installed per the authorized shop drawings may be considered as capable of carrying a load equal to the resistance value given by the equation in *Standard Specifications*, Section 49-2.01A(4)(c), *Piling – Department Acceptance*, but not more than 45 tons for timber piles. The actual nominal pile resistance must be at least twice the falsework design load, i.e. **FS = 2.0**, see *Standard Specifications*, Section 48-2.03B, *Temporary Structures – Falsework – Construction – Foundations*.

Piles which are cut off and capped near the ground line may be considered as laterally supported against buckling. See Figure 8-22, *Piles Capped Close to Ground*. The load carrying capacity of pile bents can be taken as equal to the driving resistance, but not more than the pile can carry when analyzed as a short column. See Figure 8-23, *Pile Bent*.

8-6.03 Steel Piles in Pile Bents

This manual does not include procedures for reviewing steel piles in pile bents. Therefore, engineering judgement is needed, and it is recommended that the reviewer consults DES Geotechnical Services, especially for pile bents in creeks, rivers, or bay waters. The procedures in Section 8-6.04, *Timber Piles in Pile Bents*, can be used as a guide for review of steel piles, however, these procedures were developed empirically from an evaluation of the load carrying capacity of timber piles, and therefore are not directly applicable to steel pile bents.

Evaluating the adequacy of steel pile bents involves the consideration of factors that are not subject to precise analysis; therefore, some subjective judgment is required. In view of this, the shop drawings should not be authorized until the engineer is satisfied that the design is stable under all anticipated loading conditions.

Referring to Figure 8-23, *Pile Bent*, depending on the design, some frame stiffness may be developed by the connection at the top of the pile. For example, if the piles are welded to a steel cap, the connection will be fixed; however, the degree of rotational restraint provided by the cap and the extent to which the fixed connection will influence pile stiffness are not readily determined. In view of the indeterminate nature of the problem, it is acceptable to simplify the system and assume the connection as pinned at the top when performing the frame analysis. However, if the contractor is relying on the fixed connections a more rigorous analysis is necessary.

Round steel piles are well suited as piles in pile bents because they have the same section modulus in all directions and hence have a more stable behavior when pulled into place or when loaded at various angles. H-piles, however, have a different section modulus depending upon the direction of the load, and therefore, have different stiffness depending upon the orientation of the pile. In addition, the H-piles are prone to buckling when pulled into place or when loaded at certain angles. Therefore, if H-piles are used in pile bents, the orientation must be clearly shown on the shop drawings and the installation in the field must match the shop drawings.

8-6.04 Timber Piles in Pile Bents

The load carrying capacity of timber piles in a pile bent is a function of many variable factors. For example, the type of soil, the depth at which the piles are fixed in the ground, the deviation of the piles from their theoretical position, and the contribution to system stability provided by diagonal bracing all affect the ability of timber pile bents to resist the applied loads, and all must be considered in the analysis.

Furthermore, the procedures used to evaluate pile capacity differ from those used in the analysis of other components of the falsework system because the pile analysis must consider the combined effect of vertical loads, horizontal loads, and eccentric loading conditions to ensure that allowable stresses are not exceeded.

The factors that influence pile capacity are discussed in detail in the following sections.

8-6.04A Required Pile Penetration

The procedure for timber pile bents is valid only if the piles penetrate the subsurface soils to the depth necessary to develop a point of fixity in the embedded pile. For a driven pile, the point of pile fixity is the location below the ground surface where the pile shaft may be considered as fixed against rotation when it is subjected to a bending moment.

Other factors being equal, the depth of embedment needed to develop pile fixity is a function of soil type. Soft soils require a deeper penetration than firm soils, but determining the actual penetration required is a matter of engineering judgment.

The ratio of the depth of pile penetration to the height of the pile above ground, **D/H,** is the criterion to ascertain whether a given pile is embedded deeply enough to develop a point of fixity. For the stress analysis, piles are considered fixed at the predicted depth below the ground surface when **D/H ≥ 0.75**.

When **D/H < 0.75** the piles are not embedded deeply enough to develop the fixed condition; therefore, they will rotate to a degree when loads are applied. The amount of rotation is a function of the restraint developed by the actual pile embedment. The degree of restraint decreases and rotation increases as the **D/H** ratio becomes smaller. When rotation occurs, bending stresses are reduced but overall pile capacity is reduced as well, and in a disproportionate amount.

The procedure to estimate pile capacity when the embedded length is insufficient to develop the fixed condition is discussed in Section 8-6.06, *Field Evaluation of Pile Capacity*.

As noted above, the analysis method developed by SC assumes that pile embedment is sufficient to develop the fixed condition. This is not an unreasonable assumption because, for most soil types, the penetration needed to obtain bearing will develop pile fixity as well. However, while this assumption may be true in general, it is not true in all cases; therefore, when timber pile bents are to be used, the authorization of the design is contingent on the piles actually penetrating to the depth assumed in the analysis. The condition should be noted on the shop drawings. The reason for this requirement is that a pile may achieve bearing before the fixity in the pile is achieved.

8-6.04B Point of Pile Fixity

The depth to the point of fixity is a function of soil stiffness and the diameter of the pile at the ground line. The relationship is:

$$\mathbf{y = kd} \tag{8-6.04B-1}$$

where **y** = Distance (depth) from ground line to the point of fixity

k = Soil stiffness factor

d = Diameter of the pile at the ground line

A widely accepted rule-of-thumb assumes that point of fixity is:

$$\mathbf{y} = \begin{cases} \mathbf{4d}, & \text{for medium hard to medium soft soil} \\ \mathbf{6d}, & \text{max, for soft yielding soils, such as bay mud} \end{cases} \tag{8-6.04B-2}$$

These assumptions have been verified by load tests and are satisfactory for most soil types.

Consideration may be given to raising the assumed point of fixity when piles are driven into very firm soils; however, caution is advisable because the driving of piles into any type of soil will tend to disturb the top few feet of the surrounding material.

An alternative approach uses information obtained from the Log of Test Boring (LOTB) sheets in the project plans. The average of the penetrometer readings for the portion of the log equal to the depth of pile penetration, adjusted by eliminating spikes, gives an indication of the relative soil stiffness. With this average value, a soil stiffness factor can be obtained graphically from Figure 8-24, *Soil Factor Chart*. Although this method may appear sophisticated, it does not ensure a more accurate result. As a practical approach, use of equation 8-6.04B-2 will simplify analysis without sacrificing accuracy except in the case of very soft soils.

8-6.04C Driving Tolerance

Unless the piles are driven within the tolerance shown on the shop drawings, it will be necessary to pull the top of each pile into line before setting the pile cap. Pulling the top of a pile from its driven position to its final position under the cap produces a bending moment, which must be, considered in the analysis. If the piles are appreciably out of line, the resulting bending stress may reduce pile capacity substantially.

Similarly, any deviation of the top of the pile in its final position from a vertical line through the point of pile fixity will result in an eccentric loading condition that also reduces pile capacity. Vertical load eccentricity, often referred to as pile lean, does not necessarily occur because a pile is pulled. It is an independent loading condition that occurs whenever the top of a pile in its final position under the cap is not centered around a vertical line through the point of pile fixity.

Pile pull and pile lean are independent loading conditions. Both conditions have the potential to reduce pile capacity substantially, and the adverse effect of both conditions must be considered in the design of timber pile bents. To ensure that they are considered, the *Standard Specifications*, Section 48-2.01C(2), *Falsework -- Submittals – Shop Drawings,* require the allowable driving tolerance for both conditions (maximum pull and maximum lean) to be shown on the shop drawings.

8-6.04D Soil Relaxation Factor

The force required to pull the top of a pile from its driven position to its final position under the cap causes the pile to bend, which in turn produces pressure on the soil below the ground surface. With time, the soil will yield under this pressure, allowing the pile to straighten to a degree. The yielding of the soil is called soil relaxation. As the soil yields or relaxes, it lowers the point of fixity, which in return lengthens the section of the pile above the point of fixity and reduces the bending stress proportionally.

The effect of soil relaxation is accounted for by a soil relaxation factor. The value of the soil relaxation factor in a given situation is a function of soil type and the length of time between the initial pull and application of the vertical load. These relationships are shown graphically in Figure 8-24, *Soil Factor Chart*.

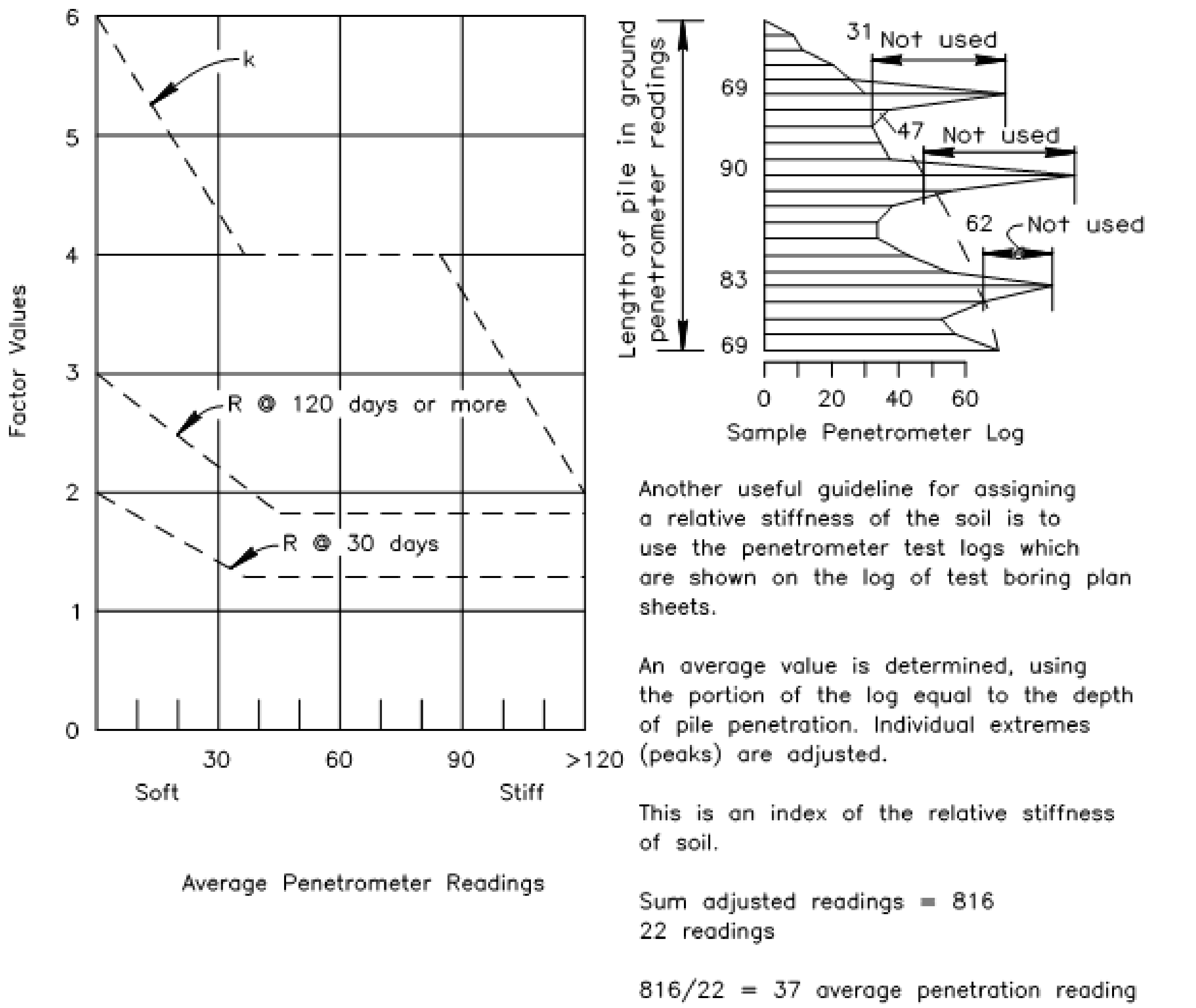


Figure 8-24. Soil Factor Chart.

For a typical bridge project, duration of time of about one month between the initial pull and application of at least a part of the vertical load is a reasonable expectation. As shown in Figure 8-24, *Soil Factor Chart*, for one-month duration of time, the soil relaxation factor is approximately:

$$\mathbf{R} = \begin{cases} \text{about } \mathbf{1.25}, & \text{for medium hard to medium soft soil} \\ \text{up to } \mathbf{2.0}, & \text{max for soft yielding soils, such as bay mud} \end{cases} \tag{8-6.04D-1}$$

If it is known ahead of time that the piles will remain unloaded for an extended period after being pulled, consideration may be given to increasing the numerical value of the soil relaxation factor. As shown on the Figure 8-24, *Soil Factor Chart*, the recommended increase is proportional to time, from 10% for 2 months up to a maximum of 50% for 4 months or longer.

8-6.04E Pile Diameter

The shop drawings must include enough information to enable the reviewer to make an independent engineering analysis. This requirement applies to pile bents as well as other elements of the falsework system. In the case of timber piling, however, the exact dimensions may not be known ahead of time. In view of this, it is customary to base the design on minimum dimensions (minimum tip and butt diameter, minimum penetration, etc.) and to show these minimum dimensions on the shop drawings.

Pile bents respond to applied loads in a different manner than other components of the falsework system. For example, if the diameter of the driven piles is larger than the diameter assumed in the analysis, vertical load carrying capacity will be increased, as will the ability of the piles to withstand the adverse effect of pile lean. However, with other factors being equal, a large diameter pile cannot be pulled as far as a smaller pile. If the bending stress caused by pulling is a significant factor in the analysis, any pile having a larger ground line diameter than originally assumed may have a lower overall load carrying capacity.

Pile diameter has a greater influence on pile capacity than any other single factor; this must be considered when selecting the value for the analysis.

8-6.05 Analysis of Timber Pile Bents

SC has adopted an empirical procedure for analysis of timber piles, which is based on the results of research involving full scale load tests on driven timber piles. The test report concludes that evaluation of pile capacity using ultimate load factors will provide a higher degree of correlation with test results than will conventional analysis using a fixed level of working stresses.

To avoid a forced compliance with working stress values that appear overly conservative in the light of falsework requirements, SC has developed a modified combined stress equation which, when used with an empirical procedure to determine the effect on pile capacity when driven piles are pulled into line, gives results that are in reasonably close agreement with the test results. Applicability of this procedure has been confirmed by mathematical analysis using a computer pile shaft program used to design pile foundations in permanent work.

The procedure is as follows:

1. Calculate the bending stress in the pile at the time the pile is pulled into position, using the maximum allowable pile pull value shown on the shop drawings. This stress is called the initial bending stress.
2. Calculate the bending stress remaining in the pile after soil relaxation has taken place. This stress is called the relaxed bending stress.
3. Calculate the bending stress caused by vertical load eccentricity, using the maximum allowable value for pile lean shown on the shop drawings.
4. Calculate the bending stress caused by the horizontal design load; calculate the lateral deflection of the pile bent and the bending stress caused by additional vertical load eccentricity resulting from that deflection. This step is not required unless **L/d > 8**. See Section 8-6.05C, *Effect of Horizontal Loads.*
5. If **L/d > 15** calculate the P- Δ deflection for the horizontal design load and for pile lean. Calculate the bending stress resulting from the P-Δ deflection. See Section 8-6.05D, *Effect of P-Δ Deflection*. (If not, skip to step 6).
6. Calculate the compressive stress in the pile.
7. Enter the appropriate values in the combined stress equation to verify the adequacy of the design.

8-6.05A Effect of Pile Pull

The procedure below outlines the steps to determine the initial bending stress that occurs when a pile is pulled and relaxed bending stress remaining in the pile after the soil has relaxed, but before the loads are applied. The bending stress caused by the initial pile pull is limited to 4000 psi.

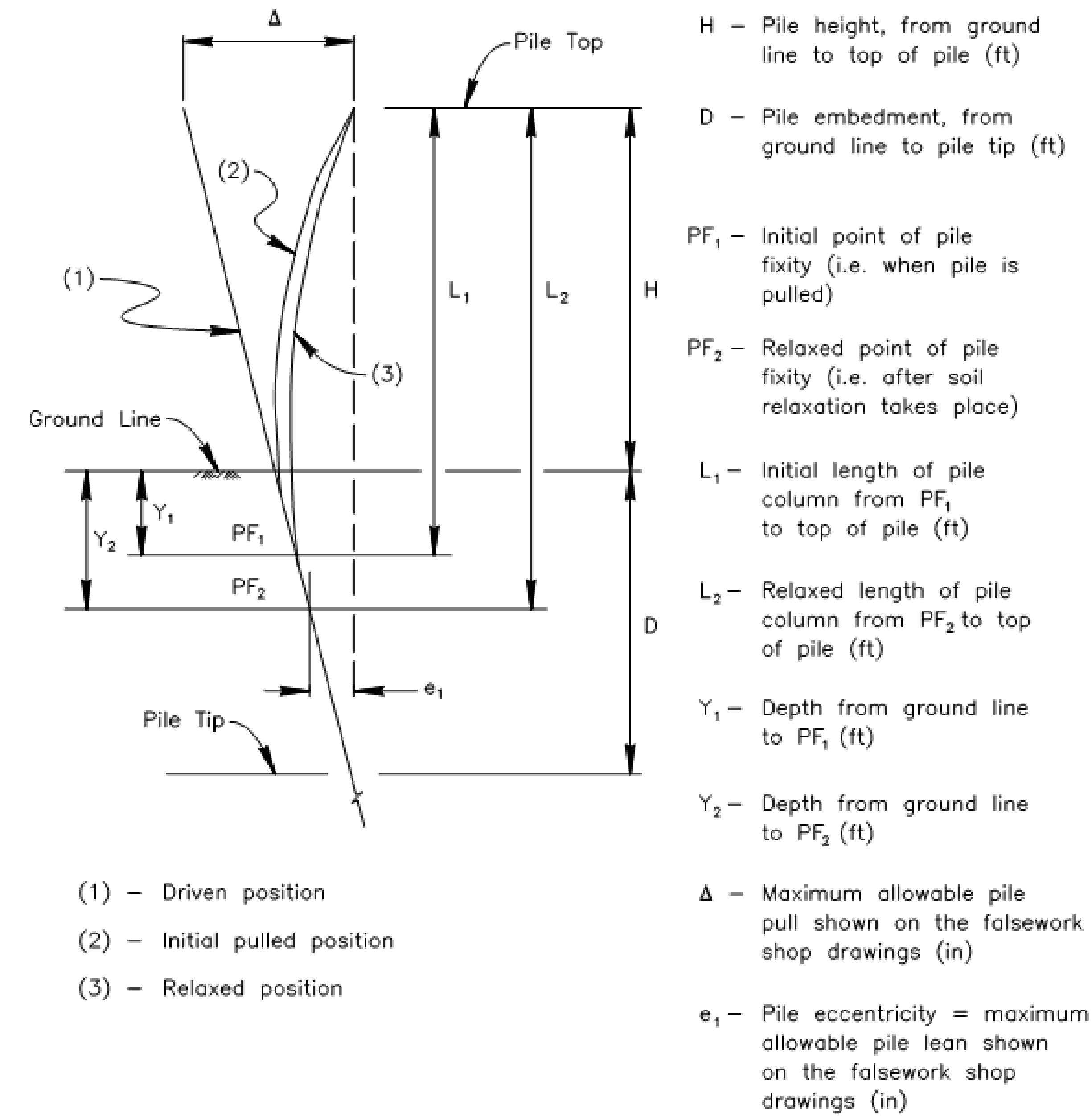


Figure 8-25. Driven Timber Pile Positions.

Refer to Figure 8-25, *Driven Timber Pile Positions,* for equation nomenclature and definition of terms used in the analysis.

The procedure is as follows:

1. Assume a ground line diameter, **d**, using the minimum butt and tip diameters shown on the shop drawings, the height of the bent from ground line to cap, **H**, and the estimated pile embedment, **D**. See Section 8-6.04E, *Pile Diameter*.
2. Using the assumed ground line diameter, **d**, from step 1, calculate the cross-sectional area, **A**, section modulus, **S**, and moment of inertia, **I.**

3. Determine modulus of elasticity, **E**, from NDS Supplement Table 6A, *Reference Design Vales for Treated Round Timber Piles Graded per ASTM D25*.
4. Determine the depth below ground line to the initial point of pile fixity, $\mathbf{Y_1}$, see Section 8-6.04B, *Point of Pile Fixity*.
5. Determine the soil relaxation factor, **R**, to be used in the analysis, see Section 8-6.04D, *Soil Relaxation Factor*.
6. Calculate the force required to pull the top of the pile from its driven position to its final position under the cap and the associated bending stress:

$$\mathbf{L_1 = H + Y_1} \tag{8-6.05A-1}$$

$$\mathbf{F_1 = \frac{3EI\Delta}{(12L_1)^3}} \tag{8-6.05A-2}$$

$$\mathbf{f_{bp(1)} = \frac{F_1(12L_1)}{S} \leq 4000psi} \tag{8-6.05A-3}$$

where $\mathbf{L_1}$ = Length of the pile from initial point of fixity, $\mathbf{PF_1}$, to top of pile when the pile is pulled initially (ft)

H = Length of the pile from the ground to the top of the pile

$\mathbf{Y_1}$ = Length of the pile from the ground to the initial point of fixity, $\mathbf{PF_1}$, when the pile is pulled initially from step 4 (ft)

$\mathbf{F_1}$ = Force required to pull the pile from its driven position to its final position (lb)

E = Modulus of elasticity from step 3 (psi)

I = Moment of inertia from step 2 (in^4)

$\mathbf{\Delta}$ = Maximum allowable distance the top of the pile may be pulled as shown on the shop drawings (in)

$\mathbf{f_{bp(1)}}$ = Initial bending stress after the pile is pulled (psi)

S = Section modulus from step 2 (in^3)

7. Calculate the force required to keep the top of the pile in its pulled position under the cap after all soil relaxation has occurred and the associated bending stress:

$$\mathbf{Y_2 = RY_1} \tag{8-6.05A-4}$$

$$\mathbf{L_2 = H + Y_2} \tag{8-6.05A-5}$$

$$\mathbf{F_2 = \frac{3EI\Delta}{(12L_2)^3} = F_1 \left[\frac{(L_1)^3}{(L_2)^3}\right]} \tag{8-6.05A-6}$$

$$\mathbf{f_{bp(2)} = \frac{F_2(12L_2)}{S}} \tag{8-6.05A-7}$$

where $\mathbf{Y_2}$ = Length of the pile from the ground to the final point of fixity, $\mathbf{PF_2}$, after soil relaxation takes place (ft)

R = soil relaxation factor from step 5

$\mathbf{L_2}$ = Length of the pile from final point of fixity to top of pile, $\mathbf{PF_2}$, after soil relaxation takes place (ft)

H = Length of the pile from the ground to the top of the pile

$\mathbf{F_2}$ = Force required to keep the pile in its final position (lb)

$\mathbf{F_1}$ = Force required to pull the pile from its driven position to its final position from step 6 (lb)

$\mathbf{L_1}$ = Length of the pile from initial point of fixity, $\mathbf{PF_1}$, to top of pile when the pile is pulled initially (ft)

$\mathbf{f_{bp(2)}}$ = Bending stress in the pile after soil relaxation (psi)

S = Section modulus from step 2 (in^3)

8-6.05B Diagonal Bracing

Pile bents are classed as either braced or unbraced depending on the degree of rigidity provided by the bracing system. In addition, a bent is considered braced if it is stabilized by external support or if the horizontal forces are carried across the bent, as is often the case in the longitudinal direction.

To be classed as a braced bent, diagonal bracing must meet the following criteria:

- Transverse bracing must comply with the provisions in Section 6-3, *Diagonal Bracing*. In addition, the frame must include a horizontal member installed in a plane through the connections at the bottom of the lowest tier of bracing. The horizontal member must be fastened to each pile in the bent with a bolted connection.
- Longitudinal bracing, if used to stabilize the bent, must comply with the criteria for transverse bracing in the preceding paragraph. If longitudinal forces are carried

across the bent, the design must comply with the criteria in Section 6-4, *Longitudinal Stability.*

8-6.05C Effect of Horizontal Loads

In a typical pile bent diagonal bracing will be installed between the cap and a point near the ground or water surface. Within the limits of a properly designed and constructed bracing system, the bracing will resist horizontal forces in the same manner as the bracing in any other framed bent. Below the bracing, however, a horizontal load will deflect the piles, and this deflection will produce a bending moment. Therefore, the ability of a pile bent to resist the horizontal design load is a function of the contribution to frame rigidity provided by the diagonal bracing and the stiffness of the individual piles.

The effect of bending stresses, produced by horizontal forces, on the stability of the system is a direct function of the unsupported length of the pile. For a diagonally braced pile bent the unsupported length is the vertical distance between the relaxed point of the pile fixity and the bolted connection at the bottom of the lowest tier of diagonal bracing.

Bending stress produced by application of the assumed horizontal load must be considered in all cases where:

$$\frac{\mathbf{L_u}}{\mathbf{d}} > \mathbf{8} \qquad \text{(8-6.05C-1)}$$

where $\mathbf{L_u}$ = Unsupported pile length (ft)
d = Diameter of the pile at the ground level (ft)

For typical pile diameters and average soil conditions, this value corresponds to a distance of about 2 feet between the ground surface and the bottom of the bracing.

8-6.05D Effect of P-∆ Deflection

When an unsupported pile is subjected to both horizontal and vertical loads, the pile will deflect laterally in the direction of the applied horizontal load. This lateral deflection moves the original point of application of the vertical load, and the resulting horizontal displacement produces an eccentric loading condition. See **x** in Figure 8-27, *Type III Timber Pile Bents Positions*.

The total vertical load eccentricity that occurs when a pile in a pile bent is deflected laterally is the sum of the deflection caused by the horizontal load and the additional deflection caused by bending which occurs as a consequence of the vertical load acting on the pile in its deflected position. The additional deflection of the pile under the applied vertical load, and the corresponding increase in the bending stress, is often referred to as the P-∆ effect.

The total deflection resulting from the combined action of a horizontal and a vertical load cannot be calculated directly since it is the sum of a converging mathematical series. However, it may be approximated by incremental addition using the iterative procedure and formulas shown in Figure 8-27, *Type III Timber Pile Positions*. See also Appendix D *Example Problems,* Example Problem 26, *Timber Pile Bents –Type III Bent*.

Additional bending due to the P-Δ effect also occurs when a vertical load is applied to an unsupported pile that is leaning in any direction. When the load is applied, the pile will deflect laterally in the direction of the pile lean. In this case the deflecting force is the horizontal component of the vertical load reaction acting along the axis of the out-of-plumb pile.

When the unsupported length of the pile is small, the lateral deflection due to the P-Δ effect will be small as well; therefore, the stress produced by additional bending in the pile may be neglected. As the unsupported length increases, however, the deflection also increases so that at some point the resulting bending stress must be considered.

Bending due to P-Δ deflection must be considered when:

$$\frac{\mathbf{L_u}}{\mathbf{d}} > \mathbf{15} \qquad \text{(8-6.05D-1)}$$

where $\mathbf{L_u}$ = Unsupported pile length (ft)

d = Diameter of the pile at the ground level (ft)

While the use of a limiting $\mathbf{L_u/d}$ ratio of 15 is considerably more liberal than is typically the case for frame analysis, this procedure is satisfactory for pile bents because of the inherent stability provided by the driven piles.

When considering the effect of P-Δ deflection, the **H** value used to begin the iterative calculation is the total horizontal force produced by the combined application of the horizontal and vertical design loads. Thus, **H** is the sum of the horizontal design load and the horizontal component of the vertical design load acting on the pile in its leaning position.

8-6.05E Braced Pile Bents

For evaluation of design adequacy, braced bents are divided into three categories, or bent types, depending on the unsupported length to diameter, $\mathbf{L_u/d}$, ratio of the pile, as follows:

Type I $\frac{L_u}{d} \leq 8$ (8-6.05E-1)

Type II $8 < \frac{L_u}{d} \leq 15$ (8-6.05E-2)

Type III $\frac{L_u}{d} > 15$ (8-6.05E-3)

As shown in the combined stress equation in the following sections, all bending stresses are additive. This occurs because, when evaluating the adequacy of a pile bent, the horizontal design load is assumed to act in the direction that produces the highest combined bending stress in the pile.

When calculating stresses and deflections in the pile, the bent will be considered as a braced frame within the vertical limits of the bracing, and the horizontal design load will be applied in a plane through the bolted connections at the bottom of the bracing.

The unsupported length of the pile is the vertical distance between the relaxed point of pile fixity and the connections at the bottom of the lowest tier of bracing. The pile is assumed to be fixed against rotation and translation at the relaxed point of fixity and free to rotate and translate with the frame at the connection at the bottom of the bracing.

The procedure in the following sections depends on the type of bent under consideration.

8-6.05E(1) Type I Pile Bents

Type I pile bents must conform to the bracing criteria in Section 8-6.05B, *Diagonal Bracing,* and satisfy equation 8-6.05E-1, $L_u/d \leq 8$.

In a Type I bent the bending stress produced by the horizontal design load may be neglected, and the modified combined stress equation is:

$$\frac{f_{bp(2)} + 2f_{be(1)}}{3F_b'} + \frac{2f_c}{3F_c'} \leq 1.0 \qquad \text{(8-6.05E(1)-1)}$$

where $f_{bp(2)}$ = Bending stress remaining in the pile after soil relaxation takes places (psi)

$f_{be(1)}$ = Bending stress due to vertical load eccentricity occurring as the consequence of pile lean (psi)

F_b' = Adjusted bending stress design value (psi)

f_c = Stress in compression parallel to the grain (axial compression) due to the vertical load (psi)

F_c' = Allowable working stress in compression (psi)

In the combined stress equation, the numerical coefficients “2” and “3” are the load factor and the working stress modification factor, respectively.

The procedure for a Type I braced bent using the modified combined stress equation is as follows:

1. Calculate the bending stress remaining in the pile, **$f_{bp(2)}$**, following the procedure explained in Section 8-6.05A, *Effect of Pile Pull.*
2. Calculate the bending stress due to vertical load eccentricity:

$$\mathbf{f_{be(1)}} = \frac{\mathbf{P_v e_1}}{\mathbf{S}} \qquad \text{(8-6.05E(1)-2)}$$

where **$f_{be(1)}$** = Bending stress due to vertical load eccentricity (psi)

P_v = Vertical design load (lb)

e_1 = Maximum pile lean shown on the shop drawings (in)

S = Pile section modulus (in^3)

3. Calculate the stress due to axial compression:

$$\mathbf{f_c} = \frac{\mathbf{P_v}}{\mathbf{A}} \qquad \text{(8-6.05E(1)-3)}$$

where **f_c** = Compressive stress due to axial compression (psi)

P_v = Vertical design load (lb)

A = Area of the pile at the ground line (in^2)

4. When longitudinal forces produced by the horizontal design load are carried across the bent, the unsupported length of the pile in the longitudinal direction, because of the absence of bracing, will be greater than in the transverse direction. In such cases it is necessary to determine the allowable compressive stress, **F_c**.
5. Enter the appropriate values and solve the combined stress equation (8-6.05E(1)-1).

8-6.05E(2) Type II Pile Bents

Type II pile bents must conform to the bracing criteria in Section 8-6.05B *Diagonal Bracing* and satisfy equation 8-6.05E-2, $\mathbf{8 < L_u/d \leq 15}$.

For Type II bents it is necessary to consider the effect of horizontal forces but not P-Δ deflection. For this case the modified combined stress equation becomes:

$$\frac{\mathbf{f_{bp(2)}+2f_{be(1)}+2(f_{bH}+f_{be(2)})}}{\mathbf{3F_b{}'}}+\frac{\mathbf{2f_c}}{\mathbf{3F_c{}'}}\leq \mathbf{1.} \qquad \text{(8-6.05E(2)-1)}$$

where $\mathbf{f_{bp(2)}}$ = Bending stress remaining in the pile after soil relaxation takes places (psi)

$\mathbf{f_{be(1)}}$ = Bending stress due to vertical load eccentricity occurring as the consequence of pile lean (psi)

$\mathbf{f_{bH}}$ = Bending stress produced by the horizontal design load (psi)

$\mathbf{f_{be(2)}}$ = Bending stress due to the additional vertical load eccentricity (psi)

$\mathbf{F_b'}$ = Allowable working stress in bending (psi)

$\mathbf{f_c}$ = Stress in compression parallel to the grain (axial compression) due to the vertical load (psi)

$\mathbf{F_c'}$ = Allowable working stress in compression (psi)

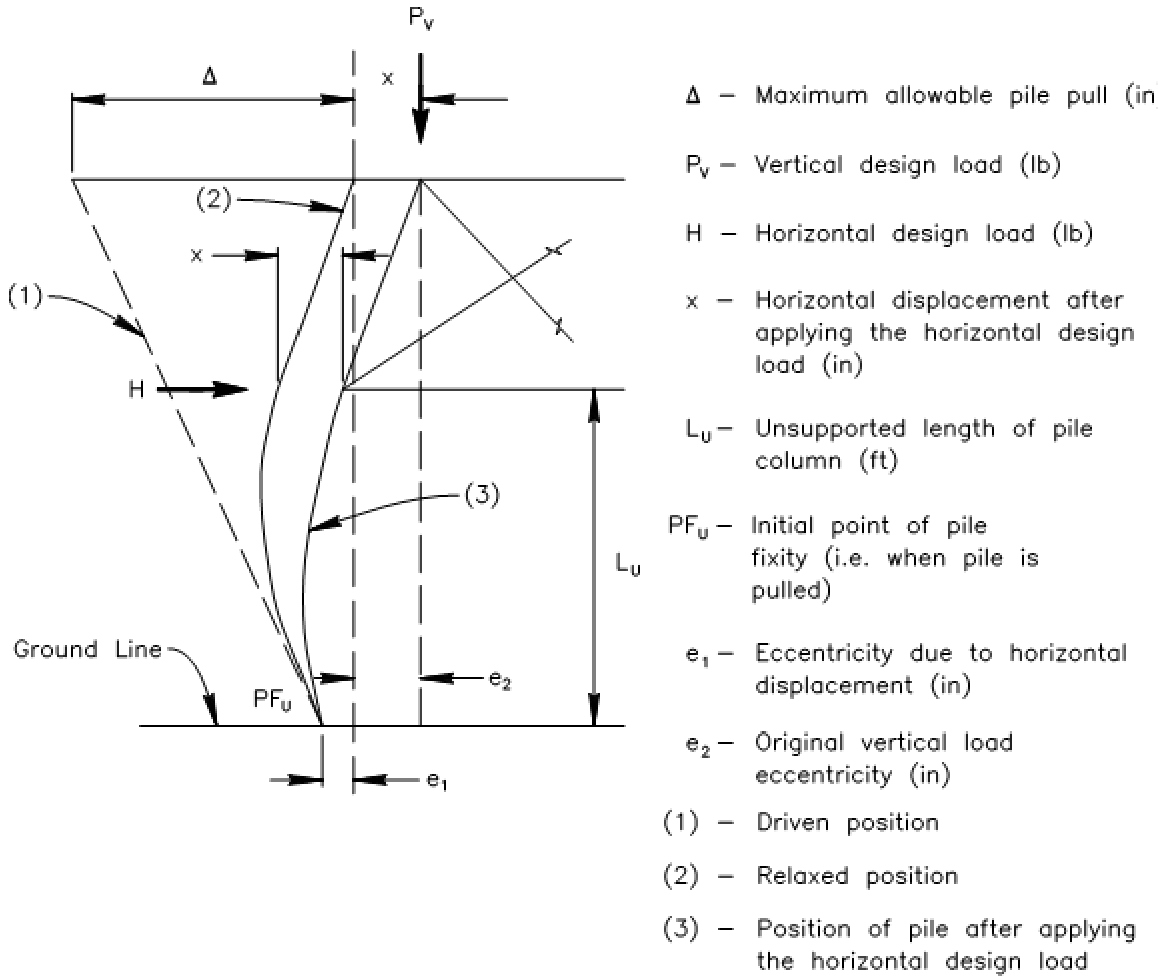


Figure 8-26. Type II Timber Pile Positions.

Figure 8-26, *Type II Timber Pile Positions,* is a schematic representation of a pile in a Type II pile bent before and after the horizontal design load is applied.

The procedure for a Type II braced bent using the modified combined stress equation is as follows:

1. Calculate the bending stress remaining in the pile, $\mathbf{f_{bp(2)}}$, following the procedure explained in Section 8-6.05A, *Effect of Pile Pull.*
2. Calculate the bending stress produced by vertical load eccentricity due to pile lean, $\mathbf{f_{be(1)}}$, using equation 8-6.05E(1)-2.
3. Calculate the stress due to axial compression, $\mathbf{f_c}$, using equation 8-6.05E(1)-3.
4. Determine the allowable stress due to axial compression, $\mathbf{F_c}$.
5. Calculate the bending stress produced by the horizontal design load.

$$\mathbf{f_{bH}} = \frac{(\mathbf{H})(\mathbf{12L_u})}{\mathbf{S}} \quad \text{(8-6.05E(2)-2)}$$

where $\mathbf{f_{bH}}$ = Bending stress due to horizontal load

H = Horizontal design load (lb)

$\mathbf{L_u}$ = Unbraced length (ft)

S = Pile section modulus (in^3)

6. Calculate the horizontal displacement that occurs when the horizontal design load is applied to the pile. See **x** in Figure 8-26, *Type II Timber Pile Positions*:

$$\mathbf{x} = \frac{(\mathbf{H})(\mathbf{12l_u})^3}{\mathbf{3EI}} \quad \text{(8-6.05E(2)-3)}$$

where **x** = Horizontal displacement (in)

H = Horizontal design load (lb)

$\mathbf{L_u}$ = Unbraced length (ft)

E = Modulus of elasticity (psi)

I = Moment of inertia of the pile (in^4)

7. Calculate the bending stress, $\mathbf{f_{be(2)}}$, due to additional vertical load eccentricity (e_2) caused by the horizontal displacement, **x**. The additional vertical load eccentricity, $\mathbf{e_2}$, is numerically equal to the horizontal displacement, **x**, see Figure 8-26, *Type II Timber Pile Positions*:

$$\mathbf{f_{be(2)}} = \frac{(\mathbf{P_v})(\mathbf{e_2})}{\mathbf{S}} \quad \text{(8-6.05E(2)-4)}$$

where $\mathbf{f_{be(2)}}$ = Bending stress due to the additional vertical load eccentricity (psi)

$\mathbf{P_v}$ = Vertical design load (lb)

$\mathbf{e_2}$ = x = Additional vertical load eccentricity caused by the horizontal displacement (in) alt text

8. Enter the appropriate values and solve the combined stress equation 8-6.05E(2)-1.

8-6.05E(3) Type III Pile Bents

Type III pile bents must conform to the bracing criteria in Section 8-6.05B, *Diagonal Bracing* and satisfy equation 8-6.05E-3, $\mathbf{L_u/d > 15}$.

For Type III bents it is necessary to consider the effect of horizontal forces and the bending stress produced by P-Δ deflection. For this case the modified combined stress equation becomes:

$$\frac{f_{bp(2)}+2f_{be(1)}+2\left(f_{bH}+f_{be(3)}\right)}{3\mathbf{F_b}'}+\frac{2f_c}{3\mathbf{F_c}'}\leq 1.0 \qquad \text{(8-6.05E(3)-1)}$$

where $\mathbf{f_{bp(2)}}$ = Bending stress remaining in the pile after soil relaxation takes places (psi)

$\mathbf{f_{be(1)}}$ = Bending stress due to vertical load eccentricity occurring as the consequence of pile lean (psi)

$\mathbf{f_{bH}}$ = Bending stress produced by the horizontal design load (psi)

$\mathbf{f_{be(3)}}$ = Bending stress due to the additional vertical load eccentricity (psi)

$\mathbf{F_b'}$ = Allowable working stress in bending (psi)

$\mathbf{f_c}$ = Stress in compression parallel to the grain (axial compression) due to the vertical load (psi)

$\mathbf{F_c'}$ = Allowable working stress in compression (psi)

The procedure for a Type III braced bent using the modified combined stress equation is as follows:

1. Calculate the bending stress remaining in the pile, $\mathbf{f_{bp(2)}}$, following the procedure explained in Section 8-6.05A, *Effect of Pile Pull.*
2. Calculate the bending stress produced by vertical load eccentricity due to pile lean, $\mathbf{f_{be(1)}}$, using equation 8-6.05E(1)-2.
3. Calculate the stress due to axial compression, $\mathbf{f_c}$, using equation 8-6.05E(1)-3.
4. Determine the allowable stress due to axial compression, $\mathbf{F_c'}$.
5. Calculate the bending stress due to application of the horizontal design load, $\mathbf{f_{bH}}$, using equation 8-6.05E(2)-2.
6. Calculate the horizontal component of the vertical load reaction, $\mathbf{H_e}$, when the vertical load (P_V) is applied to the pile in its initial leaning position:

$$\mathbf{H_e} = \frac{(\mathbf{P_v})(\mathbf{e_1})}{(\mathbf{12L_2})} \quad (8\text{-}6.05E(3)\text{-}2)$$

where $\mathbf{H_e}$ = Horizontal component of the vertical load (lb)

$\mathbf{P_v}$ = Vertical load (lb)

$\mathbf{e_1}$ = Maximum pile lean shown on the shop drawings (in)

$\mathbf{L_2}$ = Length of the pile when (P_v) is applied (ft)

7. Calculate the total horizontal force, $\mathbf{H_T}$. Both the horizontal design load, **H**, and the horizontal component of the vertical design load, $\mathbf{H_e}$, act on the pile to produce additional vertical load eccentricity. Therefore, these two forces are added to obtain the total horizontal force, $\mathbf{H_T}$, to use in the P-Δ calculation:

$$\mathbf{H_T = H + H_e} \quad (8\text{-}6.05E(3)\text{-}3)$$

where $\mathbf{H_T}$ = Total horizontal force (lb)

H = Horizontal design load (lb)

$\mathbf{H_e}$ = Horizontal component of the vertical load (lb)

8. Using the total horizontal force, $\mathbf{H_T}$, from step 7, calculate the total horizontal displacement, $\mathbf{e_3}$, following the procedure explained in Section 8-6.05D, *Effect of P-Δ Deflection,* and illustrated in Figure 8-27, *Type III Timber Pile Positions*. See also Appendix D, Example Problem 26, *Timber Pile Bents – Type III Bent.*

   Referring to Figure 8-27, *Type III Timber Pile Positions*, the value for **H** is the actual horizontal force being used in the analysis. In the equations, all horizontal force values are in pounds. The iteration may be discontinued when the calculated total displacement exceeds the previously calculated total displacement by less than 5 %.

9. Calculate the bending stress, $\mathbf{f_{be(3)}}$, produced by the horizontal displacement, $\mathbf{e_3}$, calculated in step 8:

$$f_{be(3)} = \frac{(P_v)(e_3)}{S} \quad (8\text{-}6.05E(3)\text{-}4)$$

where $\mathbf{f_{be(3)}}$ = Bending stress due to the horizontal displacement (psi)

$\mathbf{P_v}$ = Vertical design load (lb)

$\mathbf{e_3}$ = P-Δ deflection due to the combined effect of the horizontal design load and pile lean (in)

S = Pile section modulus (in^3)

10. Enter the appropriate values and solve the combined stress equation 8-6.05E(3)-1.

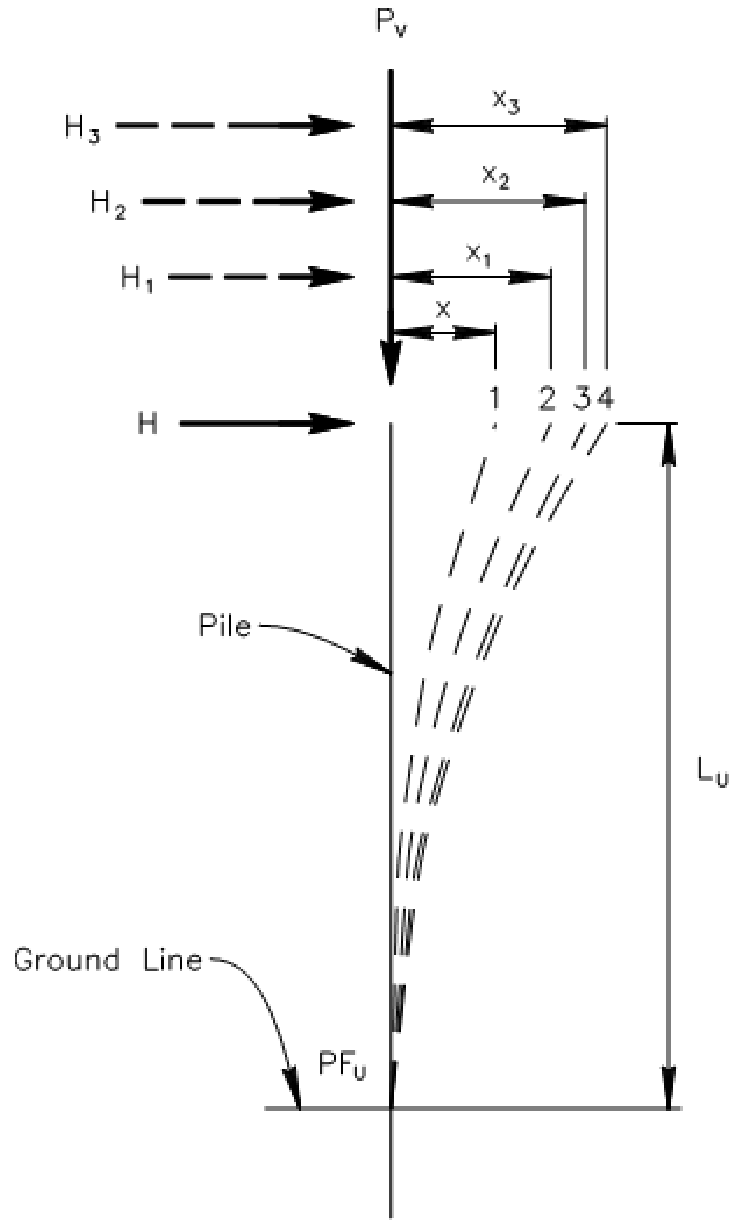


Figure 8-27. Type III Timber Pile Positions.

8-6.05F Unbraced Pile Bents

An unbraced bent is any bent where diagonal bracing is not used, and which is not stabilized by external support. The term unbraced bent also includes any braced or partly braced bent where the bracing does not meet the criteria in Section 8-6.05B, *Diagonal Bracing*.

When calculating the deflection and bending moment in an unbraced pile bent, the horizontal design load will be applied in a plane at the top of the piles, and the piles will be analyzed as unsupported cantilevers extending from the relaxed point of pile fixity to the pile cap.

Except for the point of application of the horizontal design load, the adequacy of unbraced bents is evaluated in the same manner as braced bents. Follow the procedure for the appropriate pile bent type as discussed in Section 8-6.05E, *Braced Pile Bents*.

8-6.05G Longitudinal Stability

The discussion in Sections 8-6.05E(2), *Type II Pile Bents,* and 8-6.05E(3), *Type III Pile Bents,* focuses on the procedures for Type II and Type III pile bents, respectively, when subjected to horizontal forces applied in the transverse direction, or parallel to the plane of the bracing. However, the falsework system must be capable of resisting horizontal forces applied in any direction; therefore, the pile bent analysis must consider longitudinal stability as well.

In most falsework designs, longitudinal stability is achieved by carrying the horizontal design load across the falsework bents to a point of external support, such as an abutment or column that is part of the permanent structure. Such designs must comply with the provisions in Section 6-4, *Longitudinal Stability*.

When pile bents are designed in accordance with Section 6-4, *Longitudinal Stability*, longitudinal application of the horizontal design load need not be considered. If, however, longitudinal stability is provided by some other means, such as diagonal bracing between two or more adjacent bents, the ability of the piles to resist the horizontal design load must be investigated.

Diagonal bracing used in the longitudinal direction must comply with the provisions in Section 8-6.05B, *Diagonal Bracing*, including the requirement for a horizontal member between the connections at the bottom of the bracing. The horizontal member must be sized to carry the horizontal design load as a compression member including bending from self-weight. If the member is not so designed, or if the bracing fails to comply with Section 8-6.05B, *Diagonal Bracing,* in any other aspect, the bent will be considered unbraced in the longitudinal direction.

When the longitudinal bracing is adequate, the horizontal design load will be applied in a plane through the connections at the bottom of the bracing, and the stresses and deflections in the pile below the bracing will be calculated as provided in Sections 8-6.05E(2), *Type II Pile Bents,* and 8-6.05E(3), *Type III Pile Bents,* for Type II and Type III bents, respectively. However, there are several additional factors that must be considered when investigating the longitudinal analysis, as discussed in the following paragraphs.

When the connections at the bottom of the longitudinal bracing are not located in the same horizontal plane as the connections at the bottom of the transverse bracing, the unbraced length of the pile below the bracing will be different for the longitudinal and transverse directions, and this may result in different bent types in the two directions. For example, a given bent may be Type II in the transverse direction, but because of the bracing location, the bent may be Type III in the longitudinal direction.

All bents that are connected by longitudinal bracing will deflect together when the horizontal design load is applied in the longitudinal direction; therefore, the total horizontal design load acting on the system must be apportioned between the bents. The proportioning is related to the stiffness of the bents in the longitudinal direction. Consider pile bents D-E and F-G in Figure 8-28, *Longitudinal Timber Pile Bent System*, if bents E and F require a larger diameter pile to carry the heavier vertical load. In such bents, the total horizontal load acting on the system must be proportioned between the bents in a manner that reflects the relative stiffness of the piles in each bent, rather than equally between the bents.

When the piles in two adjacent pile bents have similar properties, each bent will resist 1/2 of the total horizontal load in the longitudinal direction.

If the two adjacent bents carry different vertical loads, the horizontal loads will be different in the transverse direction for each bent and the longitudinal horizontal load will be different as well. For example, consider the bent and bracing arrangement shown schematically in Figure 8-28, *Longitudinal Timber Pile Bent System*. For braced bent D-E the horizontal design load (HDL) is:

- In the transverse direction the 2% of the total dead load is:

$$\mathrm{HDL_D} = 0.02\mathrm{P} \tag{8-6.05G-1}$$

$$\mathrm{HDL_E} = 0.02(1.5\mathrm{P}) \tag{8-6.05G-2}$$

- In the longitudinal direction the 2% of the total dead load is:

$$\mathrm{HDL_D} = \left(\frac{1}{2}\right)0.02(\mathrm{P} + 1.5\mathrm{P}) \qquad (8\text{-}6.05\mathrm{G}\text{-}3)$$

$$\mathrm{HDL_E} = \left(\frac{1}{2}\right)0.02(\mathrm{P} + 1.5\mathrm{P}) \qquad (8\text{-}6.05\mathrm{G}\text{-}4)$$

where **P** = Total dead load, see *Standard Specifications*, Section 48-2.02B(2), *Falsework – Design Criteria – Loads*.

Even where the vertical load is the same at all bents under consideration, the horizontal design load is not necessarily the same. For example, at bent A-B in Figure 8-28, *Longitudinal Timber Pile Bent System*, the horizontal design load in the transverse direction is the same for both bents. In the longitudinal direction, however, some portion of the horizontal load generated by the vertical load applied to the free-standing bent C will be carried over to bent B, and this produces a greater horizontal design load longitudinally than transversely at each bent in the A-B system.

Finally, differences in the applied vertical load on adjacent braced bents may create a situation where the piles in the two bents will have different physical properties. As an example, this would be the case at bents D-E and F-G, if bents E and F require a larger diameter pile to carry the heavier vertical load. In such bents, the total horizontal load acting on the system must be proportioned between the bents in a manner that reflects the relative stiffness of the piles in each bent, rather than equally between the bents.

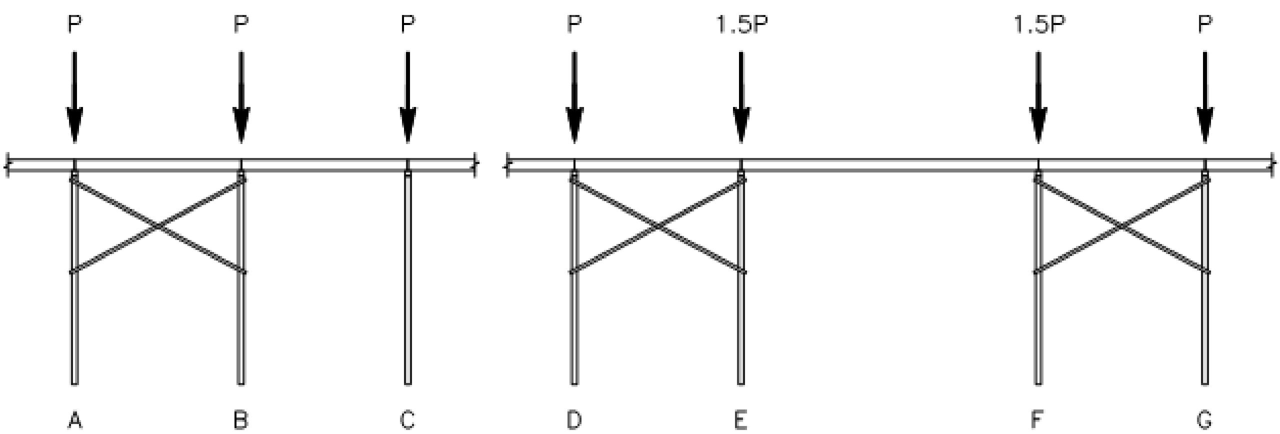


Figure 8-28. Longitudinal Timber Pile Bent System.

8-6.06 Field Evaluation of Pile Capacity

Because of the construction uncertainties associated with pile driving, piles in the driven position do not always attain the penetration assumed in the analysis. Additionally, unanticipated driving and/or site conditions may cause a driven pile to deviate significantly from its planned position shown on the shop drawings.

Contractually, any pile that fails to reach the required penetration, meet design assumptions, or deviates from its theoretical position greater than the allowable deviation shown on the shop drawings, is rejected without further evaluation because the construction work represented by that pile is not in conformance with the authorized shop drawings.

However, the contractor may submit a request for further evaluation of the rejection by submitting revised shop drawings and calculations showing that the pile does conform with the design assumptions. The revised shop drawings must be submitted, reviewed, and authorized using normal procedures.

It is emphasized that field personnel are not authorized to undertake any unilateral investigation or authorize a driven pile which does not conform to the requirements on the shop drawings.

The procedures used to estimate the capacity of piles which do not attain the penetration necessary to develop pile fixity, or which in their driven position exceed the allowable driving tolerances shown on the shop drawings, are explained in the following sections.

8-6.06A Failure to Attain Required Penetration

As discussed in Section 8-6.04A*, Required Pile Penetration*, the ratio of the depth of pile penetration to the height of the pile above the ground surface, **D/H**, is the criterion to ascertain whether a given pile is driven deeply enough to develop the fixed condition.

Pile fixity is assumed when:

$$\frac{\mathbf{D}}{\mathbf{H}} \geq \mathbf{0.7} \qquad \text{(8-6.06A-1)}$$

where **D** = Depth of pile penetration (ft)

H = Height of pile above ground surface (ft)

When driven piles do not attain the penetration necessary to assure the fixed condition, the procedure discussed in the preceding sections of this manual is not valid. However, SC has developed an alternative procedure that may be used to estimate the load carrying capacity of such piles.

The alternative procedure assumes that any pile having a ratio of the depth of pile penetration to the height of the pile above the ground surface of **D/H < 0.75**, will rotate to a degree when the loads are applied. The amount of rotation is a function of the restraint developed by the pile embedment obtained. The degree of restraint decreases, and rotation increases as the **D/H** ratio becomes smaller; therefore, the procedure depends on the actual **D/H** ratio in a given situation, as explained in the following Sections.

Figure 8-29, *Pile Rotation Stiffness Coefficient,* shows that pile rotation will reduce the relative stiffness of a pile for all **D/H < 1**, although the stiffness coefficient is too small to have an appreciable influence on pile capacity until **D/H = 0.75**. For this reason, **0.75** was selected as a practical limiting **D/H** ratio for the fixed condition assumption.

8-6.06A(1) Analysis for 0.45 ≤ D/H < 0.75

When **0.45 ≤ D/H < 0.75**, the piles can resist some bending. The amount of bending resistance developed by a given pile is an inverse function of the degree of rotation. As the D/H ratio decreases between the limiting values, rotation increases and bending resistance and overall load carrying capacity are reduced.

To account for the reduced overall load carrying capacity when rotation occurs, a stiffness reducing coefficient, **Q**, is applied when calculating the depth to the point of pile fixity. The stiffness reducing coefficient, **Q**, is obtained graphically from the Figure 8-29, *Pile Rotation Stiffness Coefficient*, which shows **Q** values for **0.45 ≤ D/H ≤ 1** for normal and soft soils.

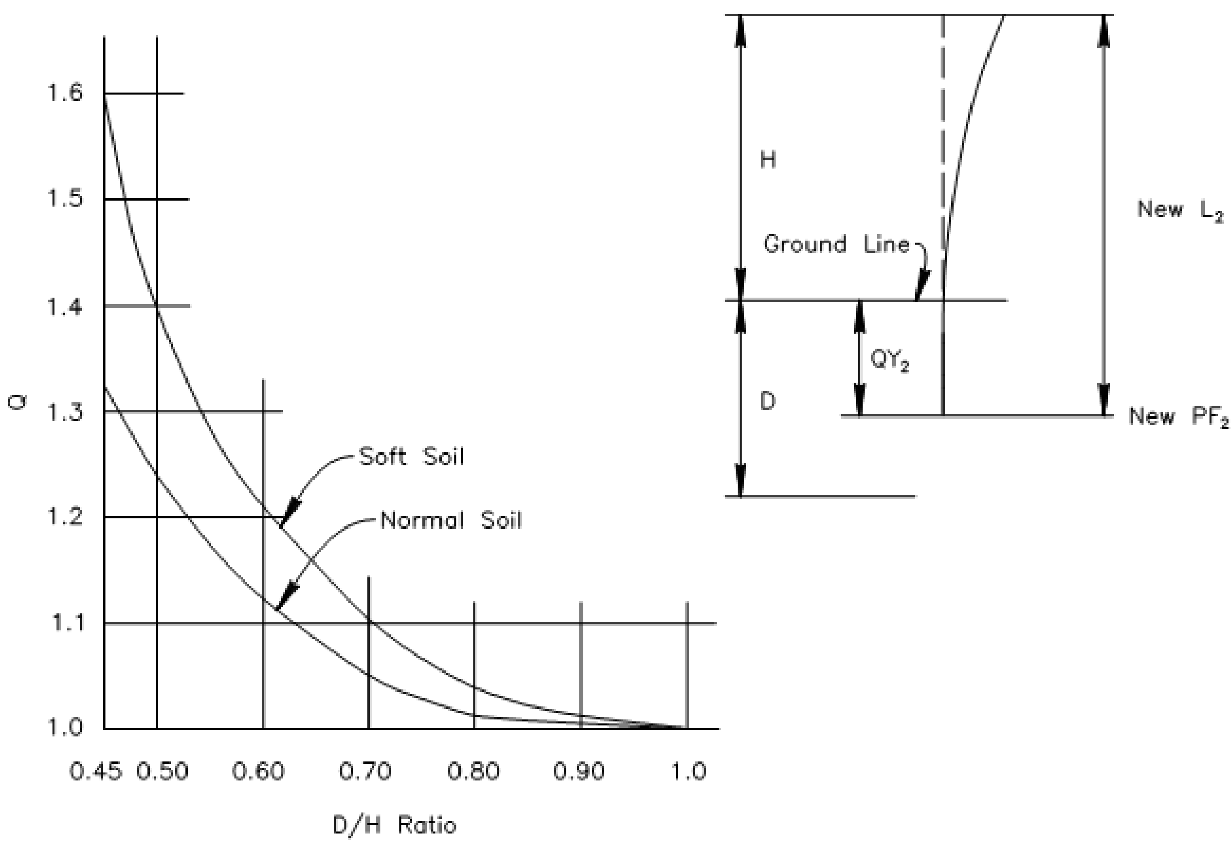


Figure 8-29. Pile Rotation Stiffness Coefficient.

The procedure for estimating pile capacity is as follows:

1. Determine stiffness reducing coefficient, **Q**, from Figure 8-29, *Pile Rotation Stiffness Coefficient*, using the **D/H** ratio of the installed pile.
2. Calculate a new length of the pile to the point of fixity, **New L_2**:

$$\textbf{New } \mathbf{L_2} = \mathbf{H} + \mathbf{QY_2} \qquad \text{(8-6.06A(1)-1)}$$

where $\mathbf{Y_2}$ = Depth from the ground line to the relaxed point of pile fixity of the properly installed pile per the original shop drawings, previously calculated using the procedure in Section 8-6.05A, *Effect of Pile Pull*, see also Figure 8-25, *Driven Timber Pile Positions.*

Q = Pile Rotation Stiffness Coefficient

$\mathbf{QY_2}$ = Depth to an adjusted point of fixity used in the analysis of the installed pile (ft)

It is unnecessary to calculate a new $\mathbf{L_1}$ value because it is unnecessary to recalculate the bending stress that occurs during the initial pile pull. The smaller **D/H** ratio results in a longer $\mathbf{L_1}$ value, which in turn produces a lower initial bending stress.

3. Calculate a new unsupported length, $\mathbf{L_u}$, and a new adjusted $\mathbf{L_u/d}$ ratio, using the new length of the pile to the point of fixity, **New L_2**. The new unsupported length, $\mathbf{L_u}$, is the vertical distance between the bottom of the bracing and the ground surface plus the depth to the adjusted point of pile fixity, $\mathbf{QY_2}$, from step 2.
4. Use the new adjusted $\mathbf{L_u/d}$ ratio to determine the pile bent type, see Section 8-6.05E, *Braced Pile Bents:*
   a. For a Type I pile bent, use the new length of the pile to the point of fixity, **New L_2**, and calculate new values for $\mathbf{f_{bp(2)}}$ and $\mathbf{f_{be(1)}}$, see Section 8-6.05E(1), *Type I Pile Bents.*
   b. For a Type II pile bent, use the new length of the pile to the point of fixity, **New L_2**, to calculate new values for $\mathbf{f_{bp(2)}}$ and $\mathbf{f_{be(1)}}$, and use the new unsupported length, $\mathbf{L_u}$, to calculate new values for $\mathbf{f_{bH}}$ and $\mathbf{f_{be(2)}}$, see Section 8-6.05E(2), Type II Pile Bents.
   c. For a Type III pile bent, use the new length of the pile to the point of fixity, **New L_2**, to calculate new values for $\mathbf{f_{bp(2)}}$ and $\mathbf{f_{be(1)}}$, and use the new unsupported length, $\mathbf{L_u}$, to calculate new values for $\mathbf{f_{bH}}$ and $\mathbf{f_{be(3)}}$, see Section 8-6.05E(3), Type III Pile Bents.

5. Enter the new values obtained in steps 4a, 4b, or 4c in the appropriate combined stress equation. The pile is adequate if the value of the equation is not greater than 1.0.

8-6.06A(2) Analysis for D/H < 0.45

For **D/H < 0.45**, the ability of a given pile to resist pullback bending decreases rapidly and, as the theoretical point of contra flexure approaches the pile tip, pile restraining capability becomes highly subjective. Furthermore, as pile embedment decreases, the type of soil has a significantly greater influence on the ability of a pile to resist rotation. When subjected to a bending moment, such piles are assumed to be free to rotate but restrained against lateral translation at or very near the pile tip.

Therefore, piles having a **D/H < 0.45** are considered as incapable of developing a true point of fixity and will only be capable of carrying axial loads. For such piles, any vertical load eccentricity and all horizontal forces must be resisted by bracing, external support, or other piles in the system.

8-6.06B Failure to Meet Driving Tolerance

Bending stresses produced by the allowable driving tolerances (pile pull and pile lean values shown on the shop drawings) are added when reviewing falsework designs for compliance with contract requirements. This procedure is a conservative approach to verify that the piles are not over stressed under the most adverse loading combination.

In practice, however, the pile pull direction may be opposite to the vertical load eccentricity caused by pile lean, in which case the adverse loading combination assumed in the analysis will not occur. When the pile pull direction is opposite to the vertical load eccentricity, the two bending stresses are compensating. Depending on the driven position, excessive pile pull in one direction may be offset by excessive lean in the opposite direction, so that the resulting combined stress is less than the allowable stress.

Figure 8-30, *Timber Pile Position,* shows that Δ and **e** are the pull and lean distances from the driven position of a pile in a braced pile bent. Both distances exceed their respective allowable values for pile pull and pile lean shown on the authorized shop drawings.

In the following general discussion, the direction of pile pull and the direction of pile lean are assumed to be in the same vertical plane.

When calculating bending stresses for the as driven position of a given pile, follow the procedures explained in Section 8-6.05, *Analysis of Timber Pile Bents*, but use the actual pile pull and pile lean distances. However, for the as-driven analysis, it is also necessary to determine whether the bending stress values are positive or negative before solving the combined stress equation.

In accordance with standard sign convention, stress values are positive or negative depending on the direction of the bending moment applied at the relaxed point of pile fixity. A clockwise moment produces positive bending stress. Conversely, a counterclockwise moment produces negative bending stress. Therefore, in a Type I bent, the combined stress equation for the general case is:

$$\left|\frac{\pm \mathbf{f_{bp(2)}} \pm \mathbf{2f_{be(1)}}}{\mathbf{3F'_b}}\right| + \frac{\mathbf{2f_c}}{\mathbf{3F_c}} \leq \mathbf{1.0} \tag{8-6.06B-1}$$

The vertical lines on either side of the bending stress fraction indicate that the absolute value of the fraction is to be used when solving the equation.

Example Problems 27, *Pile Penetration Failure – Type I Bent* and 28, *Pile Penetration Failure –Type II Bent,* in Appendix D *Example Problems,* illustrate the procedure to be followed when evaluating the load carrying capacity of driven piles.

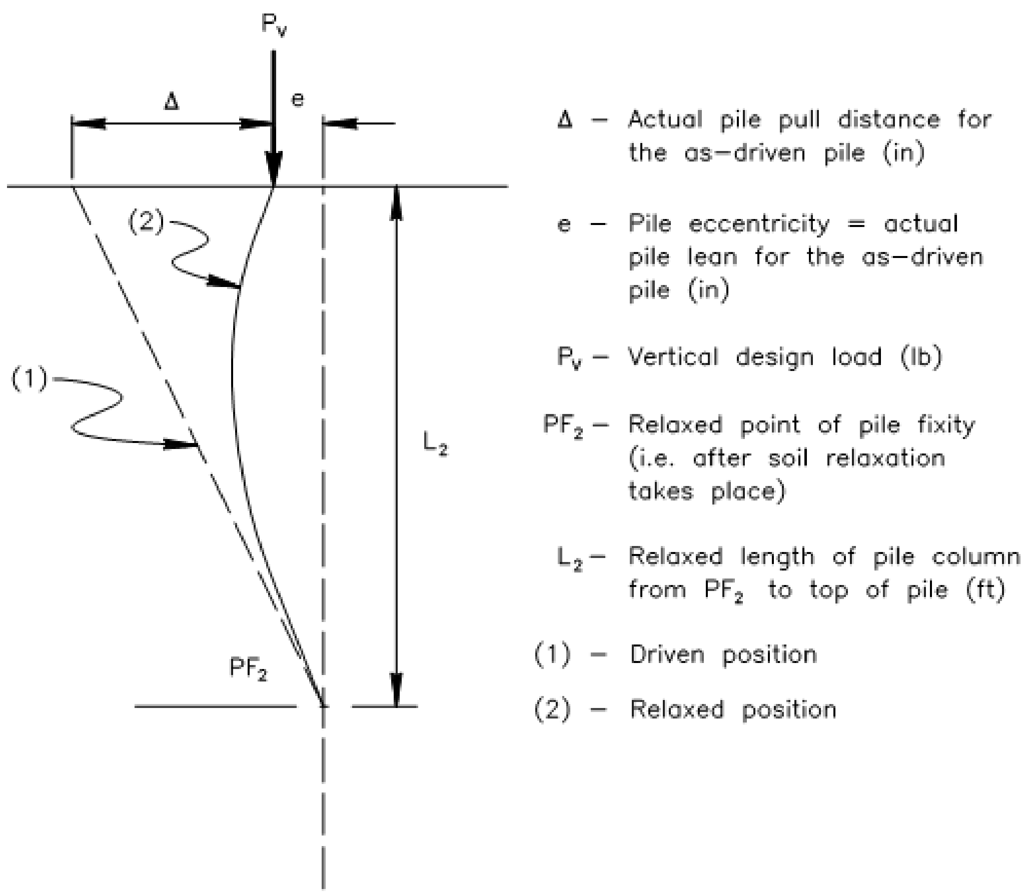


Figure 8-30. Timber Pile Position.

8-6.06B(1) Type I Pile Bents

Refer to Figure 8-30, *Timber Pile Position*, the pile pull produces a clockwise bending moment and therefore a positive bending stress, and the vertical load eccentricity due to pile lean produces a counterclockwise bending moment and therefore a negative bending stress. Hence, the combined stress equation is:

$$\left|\frac{+\mathbf{f_{bp(2)}}-\mathbf{2f_{be(1)}}}{\mathbf{3F_b}'}\right|+\frac{\mathbf{2f_c}}{\mathbf{3F_c}'}\leq \mathbf{1.0} \qquad \text{(8-6.06B(1)-1)}$$

When the position of a pile in a Type I pile bent exceeds the driving tolerances shown on the shop drawings, the capacity of that pile may be estimated as follows:

1. Calculate the initial bending stress due to pile pull, $\mathbf{f_{bp(1)}}$, using the actual pull distance. If the calculated stress is less than the allowable stress of 4000 psi for the initial pull, calculate the relaxed bending stress, $\mathbf{f_{bp(2)}}$, see equations 8-6.05A-3 and 8-6.05A-7.
2. Calculate the bending stress due to pile lean, $\mathbf{f_{be(1)}}$, using the actual eccentricity distance, see equation 8-6.05E(1)-2.
3. Determine the direction of the applied bending moment at the relaxed point of pile fixity and the sign (positive or negative) of the two bending stresses.
4. Determine the stress due to axial compression, $\mathbf{f_c}$, using equation 8-6.05E(1)-3. Axial compression is not affected by the excessive pile pull or pile lean, therefore, the value to be used in this analysis is the value calculated for the design review.
5. Enter the stress values and solve the combined stress equation using equation 8-6.06B(1)-1. The load-carrying capacity of the pile in its driven position is satisfactory if the value of the combined stress equation is not greater than 1.0.

8-6.06B(2) Type II Pile Bents

When the pile to be evaluated is in a Type II pile bent, it is also necessary to consider the effect of horizontal deflection and the combined stress equation for the general case is:

$$\left|\frac{\pm f_{bp(2)}\pm 2f_{be(1)}}{3\mathbf{F_b}'}\right|+\frac{2\left[F_{bH}+f_{be(2)}\right]}{3\mathbf{F_b}'}+\frac{2f_c}{3\mathbf{F_c}'}\leq 1.0 \qquad \text{(8-6.06B(2)-1)}$$

As shown in the equation, both the relaxed bending stress, $\mathbf{f_{bp(2)}}$, and the stress due to pile lean, $\mathbf{f_{be(1)}}$, may be either positive or negative depending on the direction of bending, while the sum of the bending stresses produced by the horizontal design load, $\mathbf{f_{bH} + f_{be(2)}}$, is positive. The bending stresses from the horizontal design load are always considered positive because, even though the horizontal design load may act from either direction, it is applied from the direction that produces the highest combined bending stress in the analysis.

8-6.06B(3) Type III Pile Bents

When the pile to be evaluated is in a Type III bent, the final term in the numerator of the bending stress fraction is replaced by $\mathbf{f_{be(3)}}$ to account for the additional vertical load eccentricity produced by P-Δ deflection.

$$\left|\frac{\pm f_{bp(2)} \pm 2f_{be(1)}}{3\mathbf{F_b}'}\right| + \frac{2\left[F_{bH} + f_{be(3)}\right]}{3\mathbf{F_b}'} + \frac{2f_c}{3\mathbf{F_c}'} \leq 1.0 \qquad \text{(8-6.06B(3)-1)}$$

8-6.06C Vector Analysis

In Section 8-6.06B, *Failure to Meet Driving Tolerance,* it was assumed that pile pull, pile lean, horizontal deflection are in the same plane. In practice, this would be an unlikely occurrence.

When the bending forces due to pile pull and pile lean act in different vertical planes, it is necessary to add the bending stress vectors geometrically and enter the resultant stress in the combined stress equation.

An analysis based on the assumption that pile pull and pile lean are in the same plane is conservative since it gives a larger combined stress equation value than an analysis that considers the direction of the bending forces. Therefore, it is not necessary to use stress vectors if the pile is acceptable using the same plane analysis.

It is a matter of engineering judgement to determine whether the relative direction of application of the bending forces is of sufficient importance to warrant vector analysis. As a guide, if the angle between the two bending planes is small, say less than about 30 degrees, same plane bending may be assumed, and the evaluation made on this basis.

If the value of the combined stress equation is less than 1.0, the pile under consideration is adequate. If the calculated value of the combined stress equation is greater than 1.0, judgment is required to determine whether reevaluation based on the direction of the applied loads using vector analysis will result in a satisfactory condition.

Generally, if the value is only slightly greater than 1.0, pile capacity should be reevaluated based on the direction of loads using vector analysis.

Figure 8-31, *Pile Vectors,* is a schematic plan view showing the:

- Location of the bottom of a pile as driven.
- Location of top of the same pile as driven.
- Location of the top of the pile after it is pulled into position under the cap.
- The direction of the pull and the direction of lean after pulling.
- Stress vector for the relaxed bending stress, $\mathbf{f_{bp(2)}}$.
- Stress vector for the bending stress due to pile lean, $\mathbf{2f_{be(1)}}$.
- Resultant of these two vectors, $\mathbf{f_{bR}}$.

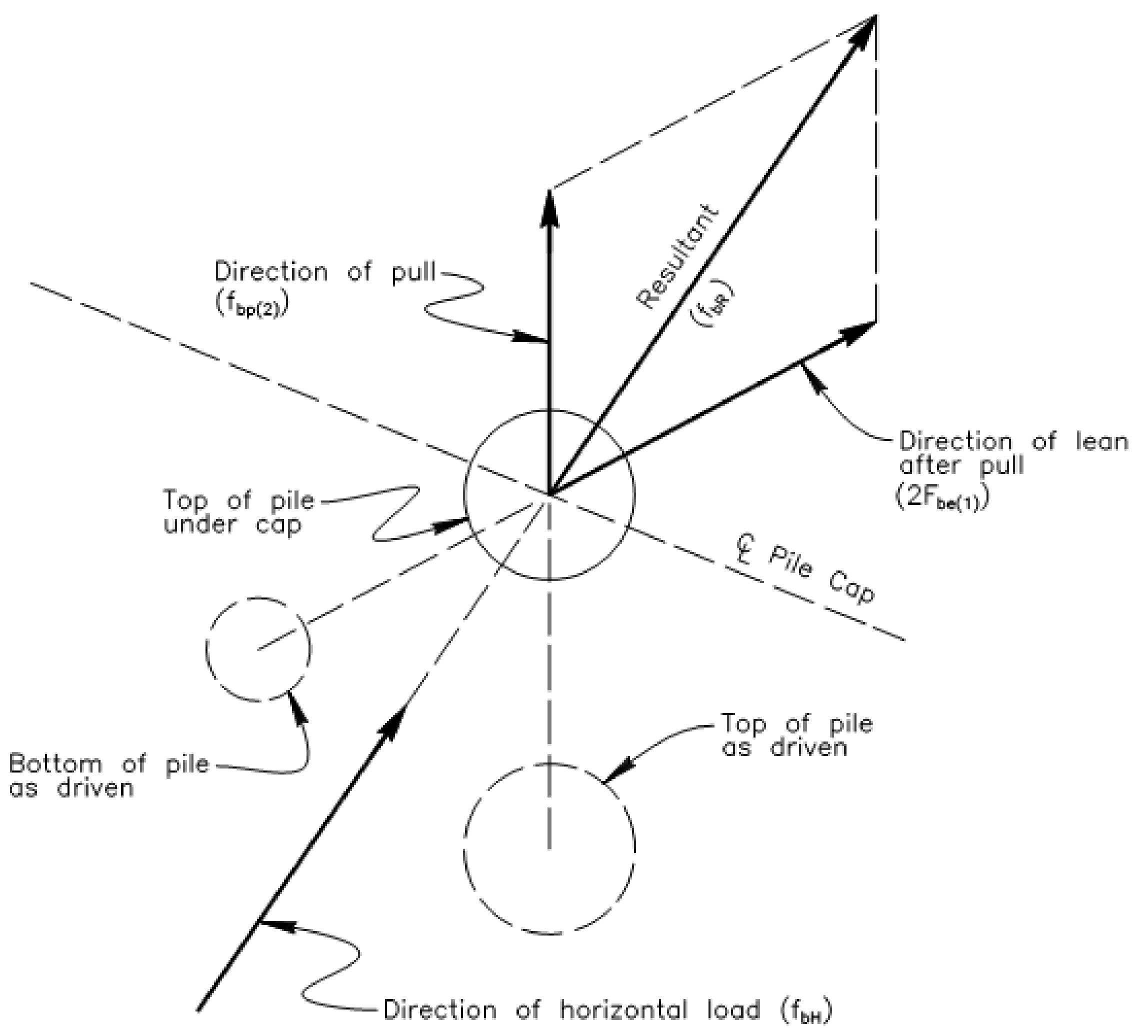


Figure 8-31. Pile Vectors.

The procedure for evaluating pile capacity using stress vectors is as follows:

1. Determine the direction of pull and the pull distance.
2. Calculate the initial bending stress due to pile pull, $\mathbf{f_{bp(1)}}$, using the actual pull distance. If the calculated stress is less than the allowable stress of 4000 psi for the initial pull, calculate the relaxed bending stress, $\mathbf{f_{bp(2)}}$, see Section 8-6.05A, *Effect of Pile Pull*.
3. Determine the direction of lean after the pile is pulled, and the magnitude of the lean.
4. Calculate the bending stress due to pile lean, $\mathbf{f_{be(1)}}$, using the actual eccentricity distance, see Sections 8-6.05E(1), *Type I Pile Bents*, 8-6.05E(2), *Type II Pile Bents*, and 8-6.05E(3), *Type III Pile Bents.*
5. Multiply the value obtained in step 4 by the load factor coefficient of 2 to obtain the stress value to use in the resultant calculation, $\mathbf{2f_{be(1)}}$.
6. Plot the stress vectors as shown in Figure 8-31, *Pile Vectors*. Plot the vectors outward from the center of the pile in the direction of pull and lean. While plotting is not essential to the calculation, it has two important advantages. First, a graphical portrayal of the problem provides a visual check on the direction and magnitude of the resultant. Second, if the vectors are plotted on a large enough scale, the resultant stress value may be scaled with sufficient accuracy to use in the remaining calculations.
7. Calculate (or scale) the resultant bending stress, $\mathbf{f_{bR}}$. Axial compression is not affected by the excessive pile pull or pile lean; therefore, it is unnecessary to recalculate the compressive stress.
8. Enter the stress values and solve the combined stress equation. The load carrying capacity of the pile in its driven position is satisfactory if the value of the combined stress equation is not greater than 1.0.

8-6.06C(1) Type I Pile Bents

For a pile in a Type I pile bent the combined stress equation is:

$$\frac{\mathbf{f_{bR}}}{\mathbf{3F_b{}'}} + \frac{\mathbf{2f_c}}{\mathbf{3F_c{}'}} \leq \mathbf{1.0} \qquad \text{(8-6.06C(1)-1)}$$

8-6.06C(2) Type II Pile Bents

For a Type II pile bent, the effect of horizontal deflection must be considered, but it is not necessary to consider the P-Δ deflection. However, since the bending stress produced by the horizontal load is not affected by excessive pull and/or excessive lean, the bending stress values to be used in the combined stress equation are the values previously calculated. Moreover, the horizontal design load may act in any direction. For analysis purposes, the horizontal design load is assumed to act in the same direction as the resultant force, $\mathbf{f_{bR}}$, because this will produce the highest stress, see Figure 8-31, *Pile Vectors*. Therefore, all bending stresses will be additive.

For a pile in a Type II pile bent the combined stress equation is:

$$\frac{\mathbf{f_{bR}+2\left(f_{bH}+f_{bc(2)}\right)}}{\mathbf{3F_b{}'}}+\frac{\mathbf{2f_c}}{\mathbf{3F_c{}'}}\leq \mathbf{1.0} \qquad \text{(8-6.06C(2)-1)}$$

8-6.06C(3) Type III Pile Bents

For a Type III pile bent, the effect of horizontal deflection and P-Δ effect must be considered. However, since the bending stress produced by the horizontal load is not affected by excessive pull and/or excessive lean, the bending stress values to be used in the combined stress equation are the values previously calculated. Moreover, the horizontal design load may act in any direction. For analysis purposes, the horizontal design is assumed to act in the same direction as the resultant force, $\mathbf{f_{bR}}$, because this will produce the highest stress, see Figure 8-31, *Pile Vectors*. Therefore, all bending stresses will be additive.

For a pile in a Type III bent the combined stress equation becomes:

$$\frac{\mathbf{f_{bR}+2\left(f_{bH}+f_{bc(3)}\right)}}{\mathbf{3F_b{}'}}+\frac{\mathbf{2f_c}}{\mathbf{3F_c{}'}}\leq \mathbf{1.0} \qquad \text{(8-6.06C(3)-1)}$$

Chapter 9: Inspection

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9-1 Introduction

The role of the field engineer during the construction phase is to provide inspection of the falsework as it is erected. The field engineer verifies that construction is in accordance with the authorized shop drawings, only sound materials are used, quality workmanship is employed, and all contract requirements are met. However, inspection by the field engineer does not relieve the contractor of his contractual responsibility for the falsework.

Timely inspection is essential. It is advantageous to verify member sizes match shop drawings while the falsework is still on the ground. Any deficiencies, such as construction which does not conform to the authorized shop drawings, poor workmanship, and use of unsound or poor-quality materials, should be brought to the contractor's attention at once. If the contractor fails to take corrective action, a non-conformance letter should be written and provided to the contractor. The letter should list the deficiencies that require remedial action, but specific corrective measures should not be ordered, nor should any predictions be made.

It is the contractor’s responsibility to comply with all Cal-OSHA requirements; however, the field engineer inspecting the falsework should be familiar with the Cal-OSHA Construction Safety Orders §1717, *Falsework and Vertical Shoring*. See Section 9-3.21, *Cal-OSHA Requirements*.

Special requirements apply to falsework over or adjacent to roadways and railroads that are open to traffic. These requirements establish a higher standard of design and construction at locations where public safety is involved. The field engineer should refer to Section 9-3.20, *Falsework Over and Adjacent to Roadways and Railroads,* to verify that the special requirements have been implemented.

9-2 Material

9-2.01 Introduction

Inspect the falsework material as it arrives at the job site. Verify material type and dimensions and verify that the material is in good condition. Reference specific members erected each day in your daily report and note the sizes and locations of the members.

9-2.02 Wood and Timber Members

Wood materials should be inspected for damage. Used wood should be examined for evidence of damage, decay, or distortion of shape; and defective or substandard pieces rejected.

Rough sawn timbers should be measured to determine their actual dimensions. Unlike surfaced material, the dimensions of rough-cut timbers are not uniform from piece to piece. The variation may be appreciable, particularly in the larger sizes commonly used for falsework posts and stringers. If the actual dimension is smaller than the dimension assumed in the design, the member may not be capable of carrying the intended load without overstress.

For a discussion on checks, shakes, and splits in wood, see *Falsework Manual,* Appendix A, *Wood Characteristics*, Section A-4.03, *Checks, Shakes, and Splits*.

Verify that the contractor has provided a certificate of compliance for all wood and timber members per *Standard Specification,* Section 48-2.01C(1), *Falsework – Submittals – General*.

9-2.02A Structural Composite Lumber (SCL)

Each piece of SCL must be marked for identification. Identification markings must include:

- Wood species
- Material grade
- Manufacturer's name or identifying symbol
- Date of manufacture or lot number
- Intended use, e.g. forming or shoring

The contractor must furnish a certificate of compliance pursuant to the *Standard Specifications,* Section, Section 6-2.03C, *Certificate of Compliance,* and Section 48-2.01C(1), *Falsework – Submittals – General,* for each delivery of structural composite lumber to the work site. The certificate must:

- Be signed by the supplier who furnishes the material. In the case of used material, the certificate must be signed by the supplier from whom the contractor originally purchased the material.

When inspecting the use of SCL in falsework pay attention to the following:

- SCL products that are designed to support wet concrete must be stored properly to protect them from direct sun light and water. When stored on the project site, they must be covered and placed on dunnage as per manufacturer’s recommendations.
- SCL must be installed per the manufacturer’s recommendations. Products may be marked for specific orientation.

- SCL should not be used as posts or diagonal bracing.

For design requirements and additional information about SCL, see Section 5-2.06, *Structural Composite Lumber*.

9-2.03 Steel Members

Steel materials should be inspected for damage. Steel members in falsework construction are typically recycled and are not in new condition. Used steel members should be examined for evidence of distortion of shape, bent flanges, holes in webs and flanges, and excessive corrosion. Defective or substandard members must be rejected.

Structural steel is commonly used in falsework construction for members such as cap beams, posts, and stringers. The steel members should be labeled with the definition of the structural shape (e.g. HP14 x 117). Flange thickness and web thickness on beams and wall thickness on posts must be measured to confirm that the dimensions match the label per the AISC Manual.

9-2.03A Used Structural Steel

Used steel members, particularly members salvaged from a previous commercial use, should be examined carefully for loss of section due to welding, rivet or bolt holes, or mechanical damage, e.g. kinks or notches in flanges, etc., all of which may reduce the load carrying capacity of the beam.

9-2.03B Welding Steel Members

Refer to *Standard Specifications,* Section 48-2.01D(2), *Falsework – Quality Assurance – Welding and Nondestructive Testing,* for welding requirements. *Standard Specifications,* Section 11, *Welding,* does not apply to welding of falsework members. Welding and inspection of steel members must comply with the following:

- All welding must comply with AWS D1.1, *Structural Welding – Steel,* welding standard.
- All welds must be performed by a certified welder. The contractor must submit a copy of the welder's certification.
- The contractor is responsible for performing an independent inspection of the welds per the AWS D1.1. This may be done by a qualified person and the inspection documented, for example in a foreman's diary. The contractor is not required to submit this documentation.
- Caltrans Materials Engineering and Testing Services (METS) can be contacted for assistance with welding.

9-2.03B(1) Welded Splices

Special requirements apply to welded splices. The contractor is required to follow the requirements in the *Standard Specifications,* Section 48-2.01D(2), *Falsework – Quality Assurance – Welding and Nondestructive Testing,* when splicing steel members by welding. These requirements include:

- The contractor must perform nondestructive testing (NDT) on welded splices using ultrasonic testing (UT) or radiographic testing (RT).
- Each weld and any repair made to a previously welded splice must be tested.
- The contractor must select locations for testing.
- The length of a splice weld where NDT is to be performed must be a cumulative weld length equal to 25% of the original splice weld length.
- The cover pass must be ground smooth at test locations.
- The acceptance criteria must comply with the specifications for cyclically loaded nontubular connections subject to tensile stress in clause 6 of AWS D1.1.
- If repairs are required in a portion of the weld, perform additional NDT on the repaired sections.
- The NDT method chosen must be used for an entire splice evaluation, including any repair.

For welded splices, the *Standard Specifications,* Section 48-2.01C(1), *Falsework – Submittals – General*, require the contractor to submit a letter of certification. The letter must be:

- Certified that all welding and nondestructive testing (NDT), including visual inspection, complies with the contract and the welding standard shown on the shop drawings.
- Signed by a civil engineer registered in the State of California.
- Submitted before any concrete is placed on the falsework being certified.

9-2.03B(2) Previously Welded Splices

The *Standard Specifications,* Section 48-1.01B, *Falsework – Definitions*, defines previously welded splices as *splices made in falsework members in compliance with AWS D1.1 or other recognized welding standard* before contract award.

For previously welded splices, the *Standard Specifications,* Section 48-2.01D(2), *Falsework – Quality Assurance – Welding and Nondestructive Testing,* require the contractor to perform and document all necessary testing and inspection required to certify the ability of the falsework members to sustain the design stresses.

For previously welded splices, the *Standard Specifications,* 48-2.01C(1), *Falsework – Submittals – General*, require the contractor to submit a welding certification. The certification must:

- Itemize the testing, inspection methods, and acceptance criteria used.
- Include tracking and identifying documents for previously welded members.
- Be signed by a civil engineer registered in the State of California.
- Be submitted before erecting the members.

9-3 Erection

9-3.01 Erection Plan

The *Standard Specifications,* Section 48-2.03A, *Falsework – Construction – General,* require the use of construction methods, which include temporary bracing when necessary, to withstand all imposed loads and to prevent overturning or collapse of the falsework during erection, construction, and removal. The means by which the contractor complies with this specification requirement during erection and construction of the falsework is commonly referred to as the "erection" plan.

Before erection begins, the erection plan, as described in the authorized shop drawings, should be discussed with the contractor's field representative who will be responsible for supervising the erection. The purpose of the discussion is to verify that the erection plan is appropriate for the site and conditions to be encountered, and that all persons involved with the work (both the contractor and owner representative) are familiar with the erection plan.

The *Standard Specifications,* Section 48-2.03, *Construction,* discusses falsework erection. The *Standard Specifications*, Section 7-1.04, *Public Safety*, prohibits erection over traffic.

9-3.02 Foundations

9-3.02A Introduction

The *Standard Specifications,* Section 48-2.03B, *Falsework – Construction – Foundations,* permit the contractor to set falsework pads and drive falsework piles before the shop drawings are authorized. However, pad placement and pile driving must be inspected, to the extent necessary to verify the adequacy of the foundation, at the time the work is done, even though the shop drawings may not yet be authorized.

Foundation layout must be confirmed before the foundation is constructed. Appropriate surveying methods should be used.

Refer to Section 9-3.20, *Falsework Over or Adjacent to Roadways or Railroads,* for horizontal clearances and necessary protection of the falsework foundation.

9-3.02B Pads

The pads are designed based on an assumed soil bearing value shown on the shop drawings. Since the pad design is based on this value, the foundation material should be inspected before the pads are set and a realistic soil bearing value assigned. Timely inspection is necessary to verify that the assumed value does not exceed the actual soil bearing value as determined by Section 8-4.06, *Soil Bearing Values,* or by a soil load test.

To provide uniform soil bearing, pads must be level and set on material that provides a firm, even surface, free of humps or depressions within the pad bearing area. To obtain uniform bearing, a thin layer of well compacted base material may be used to fill in surface irregularities.

The soil bearing capacity of some soils is greatly reduced when the ground becomes saturated. To prevent loss of support, pad foundations must be protected from flooding and undermining from surface runoff, and the pad area should be graded so the water drains away from the pads. If the pad area is flooded by a rain storm, or other event, the contractor must re-evaluate soil bearing capacity to verify that the soil can still adequately support the falsework loads as per the authorized shop drawings.

When pads are set on material backfilled around pier walls or columns in stream channels or other locations where there are no specific compaction requirements, care must be taken to verify that the backfilled material is sufficiently compacted to provide the required soil bearing value for the falsework foundation.

Benches in slopes should be cut into firm material, with the pad set back from the edge of the bench. The cut of the slope and the location of the pad must be as shown on the authorized shop drawings. The layout must comply with the requirements in the Cal-OSHA Construction Safety Order, Article 6, *Excavations*.

9-3.02B(1) Soil Load Test

The *Standard Specifications,* Section 48-2.03B, *Falsework – Construction – Foundations,* require the contractor, when requested by the engineer, to perform load tests to verify the that the design soil bearing values do not exceed the soil capacity. Therefore, the engineer should request a load test if there is uncertainty as to the ability of the foundation material to support the loads to be imposed. Section 8-4, *Soil Load Test and Soil Bearing Values,* provides information about load testing. This section should be reviewed prior to the performance of any load test to verify the adequacy of falsework foundation materials. The Division of Engineering Services (DES) Geotechnical Services is available for consultation and advice as to the suitability of a particular load test in a given field situation, as well as interpretation of test results.

9-3.02B(2) Timber Pads

Timber pads should be inspected for damage. Used timber pads should be examined for evidence of decay and distortion of shape, and defective or substandard pieces rejected.

Rough sawn timbers should be measured to determine their actual dimensions. If the actual dimension is smaller than the dimension assumed in the design, the pad may not be capable of carrying the intended load without overstress.

Continuous pads should be inspected to verify that joints are located as shown on the authorized shop drawings.

9-3.02B(3) Concrete Pads

The contractor may use the authorized concrete pads in Section 7-6.02, *Authorized Concrete Pads*, or fabricate their own pads per Section 7-6.03, *Other Concrete Pads*. Verify that the contractor has submitted a certificate of compliance as required in Sections 7-6.02C, *Certificate of Compliance,* and 7-6.03C, *Certificate of Compliance*.

Inspect the pads for damage. Examine used pads for cracks, chipping, and corrosion of the reinforcement. Defective or substandard pads must be rejected.

9-3.02C Piles

Falsework pile driving operations must be inspected to the extent necessary to verify that the required bearing values are obtained, and design assumptions are met. *Standard Specifications*, Section 48-2.03B, *Falsework – Construction – Foundations,* refers to *Standard Specifications*, Section 49, *Piling,* for pile installation.

The pile resistance value required to support the design load will be shown on the shop drawings. Piles that are plumb and properly installed per the shop drawings may be considered as capable of this resistance. The actual nominal pile resistance must be at least twice the falsework design load, i.e. **SF = 2.0**, see also Section 8-6.02, *Pile*

Resistance. Resistance values for falsework piles are determined by *Standard Specifications,* Section 48-2.03B, *Falsework – Construction – Foundations*. These specifications refer to the formula in the *Standard Specifications,* Section 49-2.01A(4)(c), *Piling – Driven Piling – Department Acceptance*. Use of the formula and inspection procedures will be the same for falsework piles as for permanent piles. The equipment required for falsework pile installation is the same as for permanent piles and is listed in the *Standard Specifications,* Section 49-2.01C(2), *Piling – Construction – Driving Equipment*.

Refer to Section 8-6.06, *Field Evaluation of Pile Capacity,* for a detailed discussion on how to address piles that are not in conformance with the shop drawings and how the contractor can resubmit revised shop drawings. It is emphasized that field personnel are not authorized to undertake any unilateral investigation or authorize a driven pile which does not conform to the requirements on the shop drawings.

Any pile that fails to reach the required penetration or deviates from its theoretical position greater than the allowable deviation shown on the shop drawings, may be rejected without further evaluation because it is not in conformance with the authorized shop drawings.

9-3.02D Pile Bents

Referring to Figure 8-23, *Pile Bent*, the design of piles in pile bents is based on certain assumptions as to penetration, driving tolerances (i.e. maximum allowable pile pull and pile lean) and the ground line pile diameter, all of which should be shown on the shop drawings.

Driving tolerances are particularly critical in pile bent designs. If little or no tolerance is permitted by the falsework design, this fact should be brought to the contractor's attention before driving begins. The orientation of H-pile cross sections must match what is shown on the shop drawings.

Falsework pile driving operations must be inspected to the extent necessary to verify that the required pile penetration and bearing values are obtained and design assumptions are met. *Standard Specifications*, Section 48-2.03B, *Falsework – Construction – Foundations,* refers to Standard Specification 49, *Piling,* for pile installation. Pile penetration cannot be verified by observation after the pile has been driven, and the distance a given pile has been pulled cannot be determined once the pile is in its final position under the cap. Section 8-6.04, *Timber Piles in Pile Bents,* includes a discussion of the assumptions that govern the design of timber pile bents, and their relative importance.

Refer to Section 9-3.20, *Falsework Over or Adjacent to Roadways or Railroads,* for horizontal clearances, necessary protection of the pile bents, and required lighting. Lighting must be installed before traffic is allowed to drive past the falsework after dark.

9-3.03 Corbels

Corbels must extend across the full width of the pads. Posts must be centered on the corbel in both directions. If other members are used to carry the post load, such as sand jacks or wedges, these members must be centered on the corbel in both directions. Verify that the spacing of the corbels complies with the shop drawings.

9-3.03A Timber Corbels

Inspect the timber corbels for damage. Used corbels should be examined for evidence of damage, decay, or distortion of shape, and defective or substandard corbels rejected.

Measure the corbels to determine their actual dimensions. If the actual dimension is smaller than the dimension assumed in the design, the corbel may not be capable of carrying the intended load without overstress.

9-3.03B Steel Corbels

Inspect the steel corbels for damage. Examine used corbels for distortion, bent flanges, or webs and holes in flanges or webs. Defective or substandard corbels must be rejected.

Measure the corbels to verify the actual structural shape. If the actual structural shape is different than what is shown on the shop drawings, the corbel may not be capable of carrying the intended load without overstress.

9-3.04 Built-Up Material

Excessive stacking of material to correct grade errors discovered during falsework construction or to accommodate short posts is an unacceptable construction practice and is not allowed. Built-up material must comply with the height to width ratio shown in Section 6-9, *Built-Up Material*.

Inspection includes, as a minimum, the following:

- Inspect the material for damage.
- Examine used material for evidence of decay or distortion of shape and reject defective or substandard material.
- Material placed on the sand jack plunger must have full bearing on the plywood plunger and must clear the frame of the sand jack by a minimum of 1/4-inch.

- Material must have full bearing and be stacked tight and neat to provide uniform bearing for the supported members.
- Verify that the built-up material complies with the shop drawings.

9-3.05 Sand Jacks

Sand jacks, which consist of compacted sand confined within a timber or metal frame, are often used to facilitate falsework removal. Typically, the sand jacks are installed on top of the corbels as shown in Figure 9-1, *Wood Sand Jacks*.

Figure 9-1. Wood Sand Jacks.

Figure 9-2. Steel Sand Jacks.

To prevent inadvertent settlement while a sand jack is still carrying a load, care must be taken to protect the sand jack from rain, flooding, or any other cause that might contribute to erosion of the sand.

The contractor has the following two options for wood sand jacks:

- Construct and use the pre-authorized wood sand jacks. See Section 7-3.01A, *Authorized Wood Sand Jack.*
- Construct sand jacks that deviate from the pre-authorized wood sand jacks and test them per Section 7-2, *Load Tests,* and 7-2.02, *Sand Jacks*. See also Section 7-3.01, *Wood Sand Jacks*.

Inspection includes, as a minimum, the following:

- Sand jacks must be new and manufactured for the current job. It is not acceptable to reuse old wood sand jacks.
- Verify that the sand jacks are manufactured by the pre-authorized details or by the authorized shop drawings.
- Sand jacks must have full bearing. A slight overhang over the supporting member is acceptable as long as the overhang is less than half the thickness of the sand jack frame.
- Plywood plunger resting on filler material (sand) must have full bearing and must clear all frame members by a maximum of 1/4-inch.
- Wood wedges or other material placed on the sand jack plunger must have full bearing on the plywood plunger and must clear the frame of the sand jack by a minimum of 1/4-inch.
- Filler material (sand) must comply with the authorized shop drawings and the filler material must be level
- Nails must be common nails. To prevent splitting, they must not be overdriven
- Quality workmanship and proper installation sufficient to bear the design load without any distress.
- Sufficient measures are in place to prevent erosion of the sand in case of rain or any other reason.

9-3.06 Wedges

Wedges are generally installed between the sand jack and the bottom cap to allow for falsework adjustment. Multiple sets of wedges (set side-by-side) are often used to keep the perpendicular-to-grain compression stresses within the allowable. Verify that the installation matches the shop drawings.

Inspection includes, as a minimum, the following:

- Inspect the wedges for damage.
- Examine used wedges for evidence of decay, or distortion of shape, and reject defective or substandard wedges.
- Wood wedges placed on the sand jack plunger must have full bearing on the plywood plunger and must clear the frame of the sand jack by a minimum of 1/4-inch.
- Wedges must have full bearing and be stacked tightly to provide uniform bearing for the supported members.
- Wedges may be used at the bottom or top of a falsework bent for adjustment, but not at both locations.

Cedar shingles, which are occasionally used as wedges, should be used with caution since cedar has a significantly lower perpendicular-to-grain strength than Douglas Fir or any of the commonly used hardwoods.

9-3.07 Bottom Caps

Refer to Section 9-3.20, *Falsework Over or Adjacent to Roadways or Railroads,* for horizontal clearances, necessary protection of the caps and mechanical connections due to impact loads.

Generally, sand jacks and wedges are placed between the corbels and bottom caps to allow for adjustment and removal of the falsework as shown in Figure 9-1, *Wood Sand Jacks*. Bottom caps must be centered on the wedges, sand jacks, and corbels. The caps must be installed level to provide a level bearing surface for the posts.

9-3.07A Timber Caps

Inspect the caps for damage. Used caps should be examined for evidence of damage, decay, or distortion of shape, and defective or substandard caps rejected.

Measure the caps to determine their actual dimensions. If the actual dimension is smaller than the dimension assumed in the design, the cap may not be capable of carrying the intended load without overstress.

9-3.07B Steel Caps

Inspect the caps for damage. Examine used caps for distortion, bent flanges, or webs and holes in flanges or webs. Defective or substandard caps must be rejected.

Measure the caps to verify the actual structural shape. If the actual structural shape is different than what is shown on the shop drawings, the cap may not be capable of carrying the intended load without overstress.

Contractors occasionally request to substitute W14 x 120 for HP14 x 117 caps. W14 x 120 may not be substituted for HP14 x 117 caps unless the authorized shop drawings specifically allow. HP14 x 117 and W14 x 120 have similar bending strength, but W14 x 120 has a thinner web and therefore is prone to web yielding or web crippling.

9-3.07C Cap Systems

Using multiple layers of caps to correct grade errors discovered during falsework construction or to accommodate short post is an unacceptable construction practice and is not allowed. If a system of multiple caps is used, the system must comply with the height to width ratio shown in Section 6-8, *Cap Systems*.

9-3.08 Traffic Braces

Refer to Figure 4-12, *Application of 2000 Lb. Load,* traffic braces are typically installed to brace the bottom caps against traffic impact loads. The braces are installed in the transverse and longitudinal directions. The braces may carry the load to the foundations or directly to the ground and must be installed per the authorized shop drawings. In the *Standard Specifications,* Section 48-2.02B(4), *Falsework – Design Criteria – Special Locations*, the term *supporting footing* means the element of the falsework system that is set on the ground.

9-3.09 Posts

9-3.09A Introduction

Refer to Section 9-3.20, *Falsework Over or Adjacent to Roadways or Railroads,* for horizontal clearances, necessary protection of the falsework members, mechanical connections due to impact loads, and required lighting. Mechanical connections must be installed before traffic is allowed under the span. Lighting must be installed before traffic is allowed to drive past the falsework after dark.

9-3.09B Timber Posts

Inspection includes as a minimum, the following:

- Inspect the posts for damage.
- Examine used posts for evidence of decay or distortion of shape, checks, or splits, and reject defective or substandard posts.
- Inspect the post before they are installed to verify that the material is acceptable. If the material is unacceptable, notify the contractor before the post is installed.
- Posts must be plumb and centered over the pad, corbel, or lower cap. Similarly, the web of the upper cap must be centered over the posts.
- Posts may be wedged at either the top or bottom for grade adjustment, but not at both locations.
- Blocking should be kept to the minimum amount needed for erection and adjustment. It is not acceptable to extend a short post by piling up blocks and wedges, since this can lead to instability.
- Full bearing must be obtained at all contact surfaces. Deficiencies in this respect may be improved by feather wedging; however, if the joint requires more than a single wedge, extra care should be taken to ensure that full bearing is obtained.

Prior to rejecting a post, several things need to be considered, such as post height, overall post quality, and bolted connection locations. Consult with the falsework reviewer for concerns about post quality. It is not unreasonable for the contractor to request banding to address checks in a post in lieu of replacing the post or adding a supplemental post. For a discussion on checks, shakes, and splits in wood, see Appendix A, *Wood Characteristics*, and Section A-4.03, *Checks, Shakes, and Splits*.

9-3.09B(1) Beam Clips

Beam clips, also known as post clips, mechanically connect steel cap beams to timber posts. The clips must be installed per the authorized shop drawings, including configuration and number of nails. The requirements for beam clips are discussed in Section 7-3.09, *Beam Clips*.

9-3.09C Steel Posts

Inspection includes as a minimum, the following:

- Inspect the posts for damage.
- Examine used posts for distortion, bent flanges or webs, and holes in flanges or webs. Reject defective or substandard posts.
- Measure the post to verify the actual structural shape. If the actual structural shape is different than what is shown on the shop drawings, the cap may not be capable of carrying the intended load without overstress.
- Inspect the posts before they are installed to verify that the material is acceptable. If the material is unacceptable, notify the contractor before the post is installed.
- Posts must be plumb and centered over the pad, corbel, or lower cap. Similarly, the web of the upper cap must be centered over the posts.
- Posts may be wedged at either the top or bottom for grade adjustment, but not at both locations.
- Blocking should be kept to the minimum amount needed for erection and adjustment. It is not acceptable to extend a short post by piling up blocks and wedges, since this can lead to instability.
- Full bearing must be obtained at all contact surfaces.

9-3.09D Aluminum Posts

Aluminum posts may be used in falsework if shown on the authorized shop drawings. However, aluminum posts are not allowed in falsework over or adjacent to roadways and railroads, per the *Standard Specifications,* Section 48-2.02B(4), *Falsework – Design Criteria – Special Locations*.

Inspection includes, as a minimum, the following:

- Inspect the posts for damage.
- Examine used posts for distortion, bent flanges or webs, and holes in flanges or webs. Reject defective or substandard posts.
- Measure the post to verify the actual structural shape. If the actual structural shape is different than what is shown on the shop drawings, the cap may not be capable of carrying the intended load without overstress.
- Inspect the posts before they are installed to verify that the material is acceptable. If the material is unacceptable, notify the contractor before the posts are installed.
- Posts must be plumb and centered over the pad, corbel, or lower cap. Similarly, the web of the upper cap must be centered over the posts.
- Posts may be wedged at either the top or bottom for grade adjustment, but not at both locations.
- Blocking should be kept to the minimum amount needed for erection and adjustment. It is not acceptable to extend a short post by piling up blocks and wedges, since this can lead to instability.
- Full bearing must be obtained at all contact surfaces.

9-3.10 Pile Bents

For installation of piles in pile bents, see Section 9-3.02D, *Pile Bents*. For bracing of pile bents see Section 9-3.12, *Bracing*.

9-3.10A Pile Friction Collars

Friction collars must conform to the requirements in Section 7-3.08, *Pile Friction Collars*. Typically, the friction collar is used to permit erection of falsework on the friction collars before the piles for flat slab bridges are cut to grade. The falsework cap will normally be set on sand jacks which sit on top of the friction collar brackets. Once the piles are cut to grade the yoke assemblies may be installed atop the pile. Friction collars may be used with or without the yoke assembly.

The use and installation of friction collars must be in conformance with the following:

- Must be installed as shown on the authorized shop drawings and used in compliance with the manufacturer's instructions.
- The collar must have full bearing on the pile.
- It is anticipated that friction collar slip will be slight after the load transfers to the yoke assembly.

- Grease may be placed on that portion of the threaded rod to be embedded in the concrete. The threaded rods in the yoke must be removed as part of the falsework removal, and the remaining holes in the structure must be finished in the usual manner.

9-3.11 Top Caps

Refer to Section 9-3.20, *Falsework Over or Adjacent to Roadways or Railroads,* for horizontal clearances, necessary protection of the caps and mechanical connections due to impact loads. Top cap webs must be centered on the posts. The caps must be installed to provide full bearing for the posts.

9-3.11A Timber Caps

Inspect the caps for damage. Used caps should be examined for evidence of damage, decay, or distortion of shape, and defective or substandard caps rejected.

Measure the caps to determine their actual dimensions. If the actual dimension is smaller than the dimension assumed in the design, the cap may not be capable of carrying the intended load without overstress.

9-3.11B Steel Caps

Inspect the caps for damage. Examine used caps for distortion, bent flanges or webs, and holes in flanges or webs. Defective or substandard caps must be rejected.

Measure the caps to verify the actual structural shape. If the actual structural shape is different than what is shown on the shop drawings, the cap may not be capable of carrying the intended load without overstress.

HP14 x 117 and W14 x 120 have similar bending strength, but W14 x 120 has a thinner web and therefore is prone to web yielding. W14 x 120 may not be substituted for HP14 x 117 unless the shop drawings specifically state so.

9-3.11C Cap Systems

Using multiple layers of caps to correct grade errors discovered during falsework construction or to accommodate short posts is an unacceptable construction practice and is not allowed.

9-3.11D Cap Beam Center Loading Strips

Center loading strips aid in transferring the vertical reaction load from stringer to cap concentrically. This prevents the stringer bottom flange from bearing on the flange edges of the cap. This is of a particular concern when stringers are placed on a steep longitudinal slope. If the stringer bears on the edge of the cap flange it can induce torsional rotation in the cap. Refer to Section 4-4, *Cap Beam Center Loading Strips,* for design considerations.

It is critical that center loading strips are centered on the web of the cap. See Figure 4-4, *Center Loading Strip Details*.

The maximum thickness of loading strips or shims must not exceed 6-inches. This limit also applies to multiple built-up strips or shims. This maximum thickness limitation eliminates excessive build-up between the cap and the stringer beam that could lead to stability problems.

9-3.12 Bracing

9-3.12A Wood Bracing

Refer to Section 9-3.20. *Falsework Over or Adjacent to Roadways or Railroads,* for horizontal clearances, necessary protection of the bracing and required connections due to impact loads.

Inspect the wood bracing for damage. Used bracing should be examined for evidence of damage, decay, or distortion of shape, and defective or substandard bracing rejected.

Measure the bracing to determine the actual dimensions. If the actual dimension is smaller than the dimension assumed in the design, the bracing may not be capable of carrying the intended load without overstress.

9-3.12B Timber Fasteners

9-3.12B(1) Introduction

Connections in timber framing for falsework bents and similar locations where engineered connections are required must be fabricated in accordance with industry guidelines as summarized in this section. See Section 5-3, *Timber Fasteners,* for the analysis of timber fasteners.

Timber fasteners must be installed per the authorized shop drawings. Verify the fastener type, number of fasteners, spacing, edge distances, and penetration.

9-3.12B(2) Nails and Spikes

Referring to Section 5-3.03B, *Required Nail Spacing*, the following governs the spacing of nails and spikes used to connect falsework bracing components:

- The average center-to-center distance between adjacent nails or spikes, measured in any direction, must not be less than the required penetration into the main member for the size of nail being used.
- The minimum end distance and the minimum edge distance in both side member and main member, must not be less than 1/2 of the required penetration.

The specific penetration requirements for nails and spikes are shown shop drawings. It is common to use a penetration of 10D, where **D** is the diameter of the fastener. Penetration requirements are discussed in Section 5-3.03A(2), *Lateral Resistance*. In most cases the penetration required to develop the design value of a given fastener can be obtained. However, when round posts are used, or the bents are skewed so that the bracing is not parallel to the side of the post, it can be more difficult to obtain the required spacing during installation. Verify that the minimum penetration is obtained, since nails or spikes having a penetration of less than the minimum will have no allowable lateral load carrying value.

9-3.12B(3) Toe-Nailed Connections

Toe-nails should be driven at an angle of approximately 30° to the member being toe-nailed, and started approximately 1/3 of the nail length, from the end of the member. See Figure 5-5, *Toe-Nailed Connection,* in Section 5-3.03C, *Toe-Nailed Connections*.

9-3.12B(4) Bolted Connections

Bolt holes and bolt installation must conform to the following:

- Bolt holes must be a minimum of 1/32-inch to a maximum of 1/16-inch larger than the bolt diameter.
- Holes in the main and side members must be aligned and the bolt centered in the hole. Tight fit requiring the forcible driving of bolts is not recommended industry practice.
- A washer or metal plate not less than a standard cut washer must be placed between the wood and the bolt head and between the wood and the nut.

Design values for bolted connections apply to bolts that have been snugly tightened. To ensure adequate strength, connections should be inspected again after the falsework is adjusted to grade and bolts retightened if necessary.

Refer to Figure 9-3, *Bolted Connection Parallel-to-Grain Loading,* and Figure 9-4, *Bolted Connection Perpendicular-to-Grain Loading*, for single fastener connections. For multiple bolt connections, see Section 9-3.12B(6), *Multiple Fastener Connections*. The end and edge distances for single bolt connections are measured from the end or side of the wood member to the center of the bolt hole, and must meet the following industry criteria for end and edge distance, where **D** is the bolt diameter:

For parallel-to-grain loading:

- Minimum end distance:
  - In tension, 7D
  - In compression, 4D
- Minimum edge distance:
  - In tension and compression, 1.5D

For perpendicular-to-grain loading:

- Minimum end distance:
  - 4D
- Minimum edge distance without load reversal:
  - 4D toward the side where the bolt is acting
  - 1.5D to the opposite edge
- Minimum edge distance with load reversal:
  - 4D to both edges. This is the case for diagonal bracing.

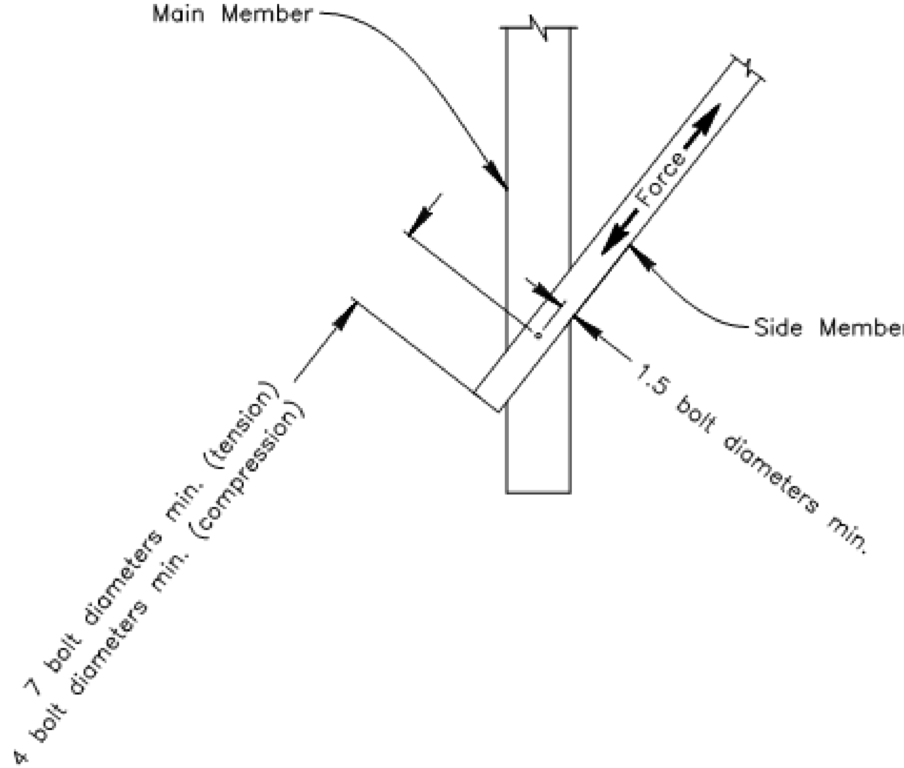


Figure 9-3. Bolted Connection Parallel-to-Grain Loading.

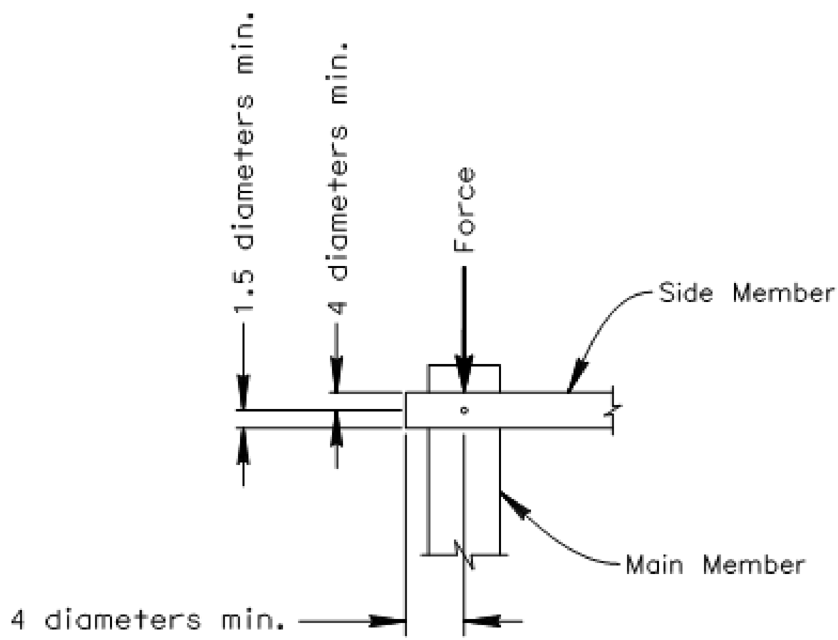


Figure 9-4. Bolted Connection Perpendicular-to-Grain Loading.

9-3.12B(5) Lag Screw Connections

Industry standards require the spacing, edge distance, end distance, and net section for lag screw connections, to conform to the requirements for bolted connections made with bolts having a diameter equal to the shank diameter of the lag screw. See Section 9-3.12B(4), *Bolted Connections*.

Lag screw installation must conform to the following:

- Insert in predrilled holes.
- The clearance hole through the first member must have the same diameter as the unthreaded shank and must go through the first member.
- The diameter of the lead hole for the threaded portion must be between 60% and 75% of the shank diameter, with the larger percentage applying to lag screws having larger diameters. The percentage range shown is for Douglas Fir Larch. For appropriate ranges for other wood species, contact the Falsework Engineer in SC Headquarters.
- The length of the lead hole must not be less than the length of the threaded portion.
- Lag screws are to be inserted in the lead hole by turning with a wrench, not by driving with a hammer.
- To facilitate installation, soap or other lubricant may be used on the screw or in the lead hole.

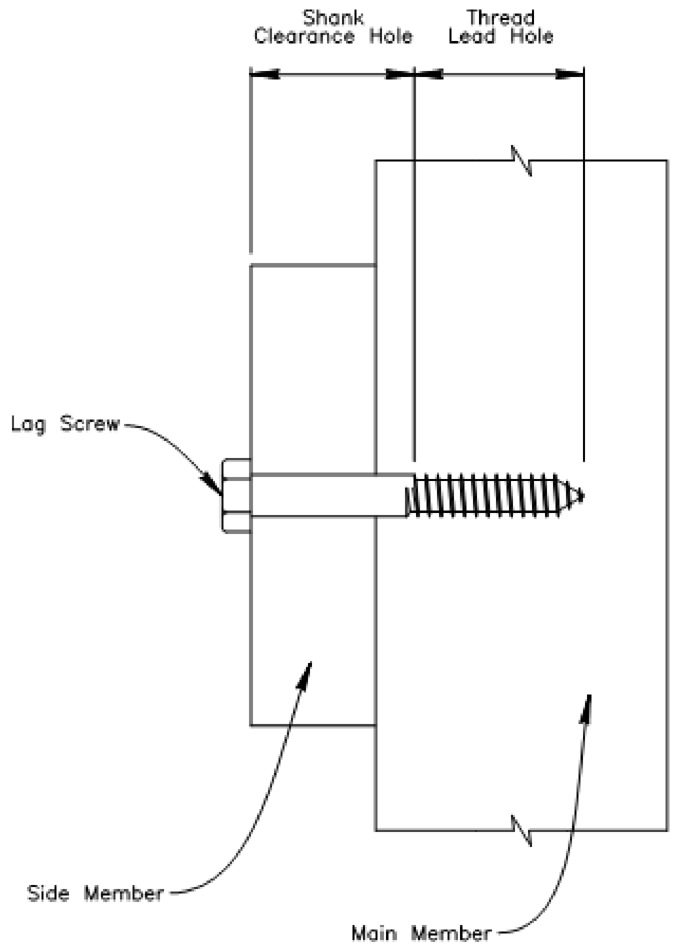


Figure 9-5. Lag Screw.

9-3.12B(6) Multiple Fastener Connections

The installation procedure discussed herein applies to both bolt and lag screw connections. In this section, the term fastener includes bolts and lag screws.

Spacing, end distance, edge distance, and the potential for wood splitting are critical for the integrity of the connection. Fastener type, diameter, length, and layout must comply with the authorized shop drawings.

Refer to Figure 9-6, *Multiple Fastener Connections*. The maximum spacing between adjacent rows of fasteners may not exceed 5 inches, regardless of other considerations.

Fastener spacing along a row is measured between the centers of adjacent fasteners, where **D** is the fastener diameter:

- For parallel-to-grain loading:
    - If the actual fastener load equals the allowable design load, the minimum spacing is 4D.

    - If the actual fastener load is less than the allowable load but not less than 75% of the allowable load, the spacing may be reduced proportionately, but not below 3D regardless of the actual fastener load.

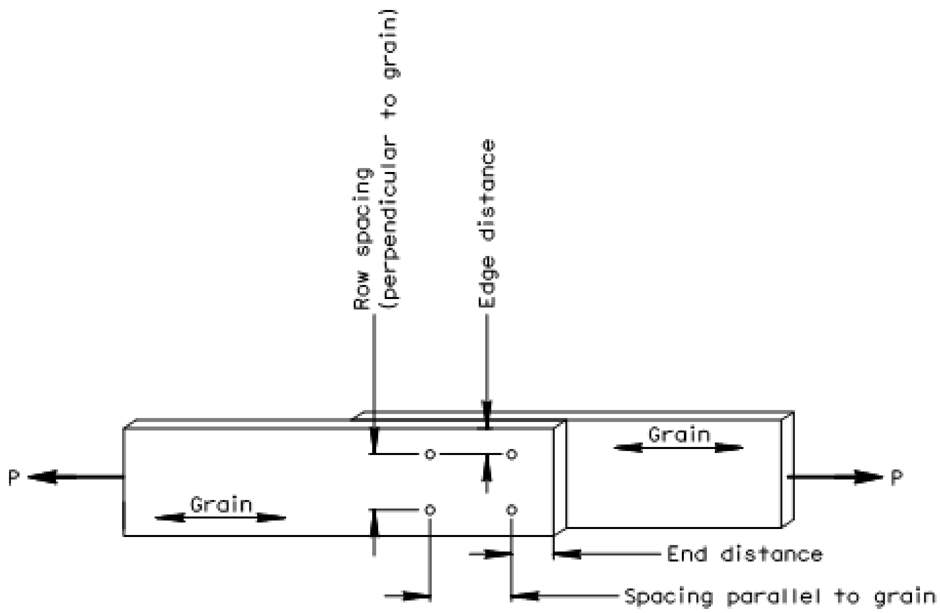
- For perpendicular-to-grain loading:
    - Spacing between fastener in a row is limited by the spacing requirements of the attached member or members loaded parallel-to-grain.

Spacing between adjacent rows is measured between the row centerlines, where **D** is the fastener diameter:

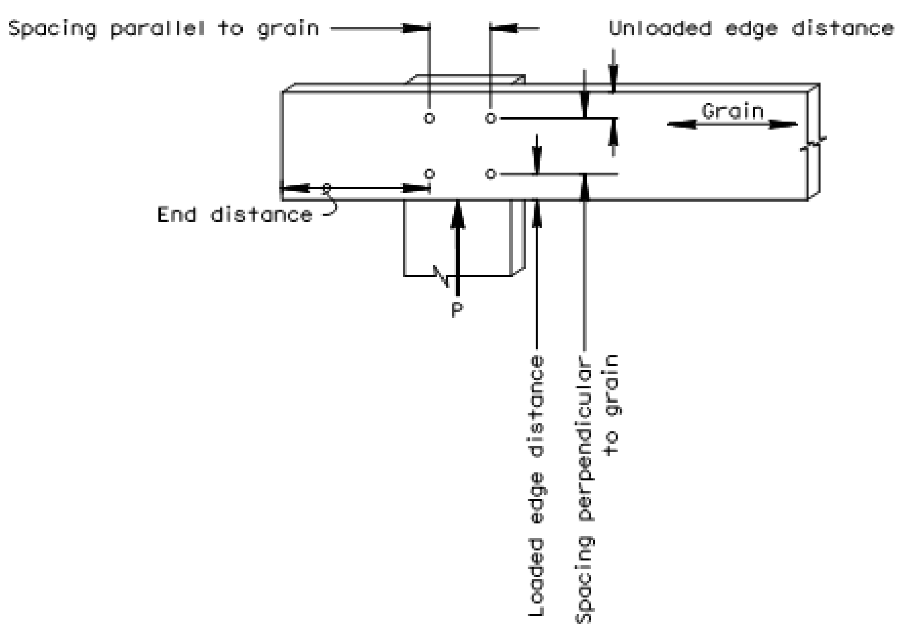
- For parallel-to-grain loading, the minimum spacing is:
    - 1.5D
- For perpendicular-to-grain loading, the minimum spacing is:
    - 2.5D for $L/D \leq 2$
    - 5D for $L/D \geq 6$
    - For $2 < L/D < 6$, the minimum spacing may be obtained by straight-line interpolation

Except as provided in the following bullet, edge and end distance requirements for multiple fastener connections are the same as the requirements for single bolt connections, where D is the fastener diameter. For single bolt requirement, see Section 9-3.12B(4,) *Bolted Connections*.

- For parallel-to-grain loading in tension or compression, the minimum edge distance is:
    - 1.5D for $L/D \leq 6$
    - 1.5D or 1/2 the distance between adjacent rows, whichever is greater for $L/D > 6$



Loading Parallel to Grain



Loading Perpendicular to Grain

Figure 9-6. Multiple Fastener Connections.

When a multiple fastener connection is loaded at an angle to the grain, as is the case with falsework bracing, industry practice requires that the gravity axis of each members in the connection must pass through the center of resistance of the fastener group to insure uniform stress in the main member and a uniform distribution of load to all fasteners. (See NDS 12.6.2).

9-3.12B(7) Drift Pins and Drift Bolts

Drift pins and drift bolts for timber are to be driven into predrilled holes having a diameter l/16-inch less than the diameter of the drift pin or drift bolt to be installed. Drift pins and drift bolts are rarely used in falsework.

9-3.12C Steel Bracing

Steel bracing can consist of structural shapes such as angles, C-channels, or hollow sections. Rebar is sometimes used for bracing, but due to its minimal stiffness it is only considered as a brace in tension.

Inspect the bracing for damage. Examine used members for distortion, bent flanges or webs, and holes in flanges or webs. Defective or substandard members must be rejected.

Measure the bracing to verify the actual structural shape. If the actual structural shape is different than what is shown on the shop drawings, the brace may not be capable of carrying the intended load without overstress.

9-3.12C(1) Steel Fasteners

Bolted connections must comply with the authorized shop drawings. Verify the fastener type, number of fasteners, spacing, and edge distances.

9-3.12D Cable Bracing

9-3.12D(1) Introduction

Refer to Section 5-5, *Cable Bracing Systems,* for a detailed discussion on the design and application of cable bracing. Cable bracing systems may be used to resist both overturning and collapsing forces. Cable systems are effective in resisting the overturning of tall falsework. They are also commonly used as diagonal bracing to resist collapse of falsework bents. Moreover, cable is also used extensively as temporary bracing to stabilize falsework bents while they are being erected or removed.

As noted in Section 5-5.01, *Introduction*, the guidance provided herein can be used when prestressed strands are used for bracing.

9-3.12D(2) Cable Inspection

Cable bracing systems require thorough inspection to ensure that the type of cable used, and the field installation conform to the details shown on the authorized shop drawings. Prior to installation, the cable should be examined to verify that the size and type of the cable and its condition (new or used) is consistent with design assumptions.

Used cables must be in serviceable condition. Used cables in serviceable condition must comply with all the requirements for rope inspection in the current edition of the *Wire Rope User's Manual*, published by the Wire Rope Technical Board. Used cable should be inspected for strength reducing flaws. The use of worn cable should not be permitted. Cable inspection includes, but is not limited to the items below:

- Diameter reduction
- Corrosion
- External wear
- Internal wear
- Kinks
- Fraying
- Protruding core
- Peening
- Scrubbing
- Broken wires

Cables must be looped around an appropriately sized thimble or equivalent diameter steel pin as recommended by the cable manufacturer.

An exception is provided for cables looped around steel caps. Cables may be looped perpendicularly around steel caps provided that appropriately sized corner softeners are used.

An exception is also provided for cables looped around timber caps where wood crushing will form an adequate radius for the cable connection.

Table 9-1, *Thimble Diameters,* may be used to determine the required thimble diameter for a given cable size:

Table 9-1. Thimble Diameters.

Cable Diameter	Approximate Standard Thimble Diameter
(in)	(in)
1/4	11/16
3/8	15/16
1/2	1-1/8
5/8	1-3/8
3/4	1-5/8
7/8	1-7/8
1	2-1/2

Cables looped around thimbles or around an equivalent diameter anchoring device are usually connected to the working part of the cable by Crosby type wire rope clips. Clip installation should be carefully inspected, since properly installed clips are critical to the effectiveness of a cable system. Proper method of installing Crosby clips is discussed in Section 9-3.12D(4), *Cable Connectors*.

To ensure adequate holding strength, field engineers should review the clip installation procedure recommended by the manufacturer before work begins. The contractor should be requested to furnish technical information from the manufacturer showing the installation procedure, recommended torque values, and other pertinent data prior to beginning erection of any cable system.

Only forged clips must be used as connectors. Forged clips are marked *forged* to permit positive identification and have the appearance of galvanized metal. Malleable clips must not be used as connectors. Malleable cable clips appear smooth and shiny.

The method by which the cable will be attached to the falsework and the location of attachment will be shown on the shop drawings. No deviation is permitted.

When cables are released for grading or adjustment, pork-chops and come-alongs, or similar systems, must be used to control the release of the cable. Loosening the clips without control is not acceptable.

9-3.12D(3) Preloading Cable

After assembly, all cable units must be preloaded to remove any slack in the cable and connections. Preloading is necessary to ensure that the cable units will act elastically when the loads are applied.

The required preload values for all cable units will be shown on the shop drawings.

Applying the preload force is an essential part of the cable system installation, and the contractor must provide a means to verify or demonstrate that the required preload force has been applied. A method used by some contractors determines the preload force by measuring the elastic elongation within a short length of the cable. Measurements are made between tape bands placed around the cable to be preloaded. Measurements between the tape bands should be done after removal of any initial slack and again after the cable unit has been preloaded. The term "initial slack" refers to excessively large loops at the connections or any excessive drape remaining in the cable after installation. The initial slack must be taken up before the preload force is applied.

When this procedure is used, the elongation calculation must be based on the reduced value of **E**, since the preload force represents only a small percentage of the cable strength. In addition, unless a pre-stretched cable is being used, constructional stretch may be a factor for consideration, see Section 5-5.09B, *Cable Preload*.

The contractor may employ other methods to demonstrate that the correct preload force is being applied; however, the method must be accurate, readily verifiable, and must not rely on subjective considerations. Regardless of the method used, measurements to verify preload values are to be performed by the contractor in the presence of the engineer.

All cable units must be preloaded simultaneously to prevent frame distortion as the preload force is applied.

Preload tensioning devices must provide positive grip so that no cable movement will occur after final tensioning. Preloading can be done with turnbuckles or with come-alongs.

When cables are attached to timber members with an appropriate fastening device, the preload force must be applied twice. The first tensioning will permit the cable fastening device to bite into the wood. Following this initial tensioning, the cable should be unloaded and then re-tensioned to the required preload force. Any additional wood crushing at the point of attachment will be minor.

Since preload force and cable drape are proportional for a given cable system, knowing the expected cable drape over a range of preload values gives the engineer a method by which the actual applied preload force may be approximated by visual inspection after the falsework is erected. For example, assume that for a particular cable a preload force of 500, 1000, and 1500 pounds results in a calculated drape of 1-1/2, 3/4, and 1/2-inches, respectively. From the relationship between drape and preload force, the engineer can readily determine the preload force actually applied.

9-3.12D(4) Cable Connectors

Cable connectors must be installed in accordance with the requirements shown on the shop drawings.

The installation of cable connectors must conform to the manufacturer's requirements. Only forged clips must be used as connectors. Forged clips are marked *forged* to permit positive identification and have the appearance of galvanized metal. Malleable clips must not be used as connectors. Malleable cable clips appear smooth and shiny.

If U-bolt (Crosby type) wire rope clips are used as connectors, the number used and the spacing is shown on the shop drawings and must conform to the data shown in Table 5-2, *Number and Spacing of U-Bolt Wire Rope Clips*.

The only correct method of attaching U-bolt wire rope clips to rope ends is shown in Figure 9-7, *Applying Wire Rope Clips*. The base (saddle) of the clip bears against the live end of the rope, while the "U" of the bolt presses against the dead end. A useful method of remembering this is: "You never saddle a dead horse."

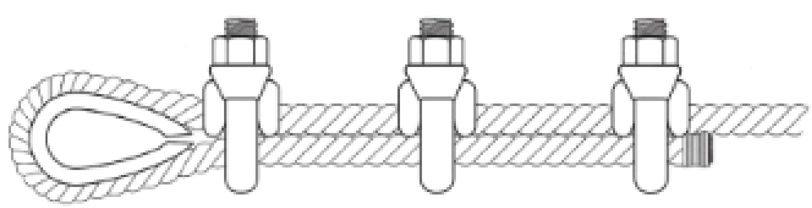


Figure 9-7. Applying Wire Rope Clips.

The clips are usually spaced about six rope diameters apart to give adequate holding strength. A wire rope thimble should be used in the loop eye to prevent kinking when wire rope clips are used. The correct number of clips for safe application, and spacing distances are shown in Table 5-2, *Number and Spacing of U-Bolt Wire Rope Clips,* in Section 5-5.04, *Cable Connector Design*.

Although the efficiency factor for Crosby clips is 80%, this value is valid only when the clip is properly installed and torqued in accordance with the manufacturer's recommendation. Tests to system failure have shown that clips that are not properly torqued will slip before the cable breaks.

9-3.13 Stringers

Refer to Section 9-3.20, *Falsework Over or Adjacent to Roadways or Railroads,* for vertical clearances and mechanical connections. It is not acceptable to install stringers so that the stringer rests on the edge of the cap. If the stringers are sloped longitudinally or if loading strips are used, see Section 9-3.11D, *Cap Beam Center Loading Strip*.

9-3.13A Timber Stringers

Inspect the stringers for damage. Used stringers should be examined for evidence of damage, decay, or distortion of shape, and defective or substandard caps rejected.

Measure the stringers to determine their actual dimensions. If the actual dimension is smaller than the dimension assumed in the design, the stringers may not be capable of carrying the intended load without overstress.

9-3.13B Steel Stringers

Inspect the stringers for damage. Examine used stringers for distortion, bent flanges or webs, and holes in flanges or webs. Defective or substandard caps must be rejected.

Measure the stringers to verify the actual structural shape. If the actual structural shape is different than what is shown on the shop drawings, the stringers may not be capable of carrying the intended load without overstress.

W14 x 120 and HP14 x 117 have similar bending strength, however, substituting one for the other is not acceptable unless the shop drawings specifically allow.

9-3.13B(1) Steel Beam Banding

Many contractors use commercial steel banding material wrapped around pairs of adjacent cross braced beams. Steel banding is less expensive and easier to install and remove than other types of tension components. However, banding is not effective unless it is properly installed and tightened. When banding is used as part of a flange support system, some means must be provided to prevent an abrupt bend or kink at the point of contact with the outer edge of the beam flange. This is an important consideration because any kink or sharp bend in commercial banding is, potentially, a point of stress concentration, which can reduce the strength of the banding material. The use of softeners, such as PVC pipe, will reduce this stress concentration, see Figure 5-11, *Two-Stringer Cross Bracing.*

Steel banding is a commercial product. If there is any question as to the adequacy of banding installed in a given situation, the contractor should be required to furnish the manufacturer's catalog data and instructions. Bracing, blocking, steel banding, ties, etc., required for lateral support of beam flanges must be installed at right angles to the beam. Bracing in adjacent bays should be set in the same transverse plane, if possible. If, because of skew or other considerations, it is necessary to offset the bracing in adjacent bays, the offset distance should not exceed twice the depth of the beam

9-3.13B(2) Stringer Connector

Stringer connectors must conform to the requirements in Section 7-3.07, *Stringer Connectors,* and Figure 7-10, *Stringer Connector Details*. The use and installation of stringer connectors must be in conformance with the following:

- Clip, bolt, and band details must match what is described in Section 7-3.07, *Stringer Connectors*
- Hole in upper part of the connector must be rounded to prevent the band from kinking when tensioned
- Bolt must bear against the stringer flange
- Angle between banding and stringer web must not exceed what is shown on the shop drawings

9-3.13C Beam Hangers

Refer to Section 7-3.03, *Beam Hangers,* and Figure 7-4, *Beam Hangers*. Beam hangers are hardware items which are placed transversely across the top flange of a beam or girder. Steel rods or bolts, which are inserted into threaded wire loops at the hanger ends, hang vertically to support the deck slab falsework, or diagonally outward to support a deck overhang bracket. Beam hangers are also commonly used for stay-in-place steel deck forms.

Verify that the beam hangers are installed per the shop drawings and used in compliance with the manufacturer’s instructions.

9-3.13D Sleepers

When falsework beams are considerably longer than the actual falsework span, the beam cantilever extending beyond the point of support will deflect upward as the main span is loaded. A filler strip (sleeper) is usually installed on the main span only, which allows free movement of the beam cantilever. The sleeper ends at the center line of the falsework top cap and does not extend into the cantilever section. The sleeper must be thick enough to offset the theoretical beam uplift on the cantilever. See Figure 4-1, *Sleeper on the Falsework Beam*.

Sleepers are also used when the bridge has a steep cross slope. In this situation the sleeper provides clearance between the joist and the stringer top flange. See Figure 4-2, *Camber Strip and Sleeper Requirements*.

9-3.13E Camber Strips

The engineer orders the contractor to furnish camber strips. See *Standard Specifications,* Section 48-2.03C, *Falsework – Construction – Erection*. Proper installation of the camber strips is important to achieve proper loading of the falsework beam. If camber strips are placed away from the centerline of a steel stringer they may induce torsional stresses that were not considered in the design.

Camber strips and their installation must conform to the following:

- 1.5-inch minimum width
- Must be centered along the longitudinal centerline of the falsework beam
- The minimum height of the camber strip must be such that the joists will not come into contact with any part of the falsework beam under any loading condition.
- Must not extend onto the unloaded portion of a trailing beam cantilever
- If the amount of camber is large, as in the case where a parabolic curved bridge soffit is supported by a long falsework beam, the camber strips should be braced or built up with wide material to avoid lateral instability. The use of laterally unsupported tall, narrow camber strips is not permitted.
- Sleepers are required when the stringer does not follow bridge cross slope and camber strips do not include allowance for cross slope, see Figure 4-2, *Camber Strip and Sleeper Requirements,* in Section 4-2.02A, *Camber Strips*.

9-3.14 C-Clamps

Refer to Section 7-3.06, *C-Clamps,* and Figure 7-7, *C-Clamps*. Heavy-duty commercial or non-commercial C-clamps having a torque-tightening capacity of 90 ft-lb or more may be used as connecting devices in accordance with the criteria in this section.

9-3.14A Commercial C-Clamps

Commercial C-clamps must conform to the requirements in Section 7-3.06A, *Commercial C-Clamps*, and the contractor must furnish a catalog or manufacturer's technical data sheet describing the clamp in sufficient detail to verify compliance with product criteria listed in this section.

9-3.14B Non-Commercial C-Clamps

Non-commercial C-clamps must conform to the requirements in Section 7-3.06B, *Non-Commercial C-Clamps,* and Figure 7-9, *Non-Commercial C-Clamp*.

9-3.14C Installation

The use and installation of C-clamps must be in conformance with the following:

- Must be installed as shown on the authorized shop drawings
- Must be torqued to 90 ft-lb
- All flanges, angle legs, plates, etc. to be connected must have constant thickness.
- Must be installed on the span side, not on the tail end of beams or stringers
- Clamps used to connect continuous steel stringers to steel caps are to be placed on the most heavily loaded span side of the cap.

9-3.15 Joists

Wood and steel joists may be used for falsework construction. If the bridge has a steep cross slope, a sleeper may be required for clearance between the joist and the stringer top flanges, see Section 9-3.13D, *Sleepers.*

Blocking of joists must be installed per the authorized shop drawings. See also Section 5-2.04F(3), *Blocking to Prevent Rollover.*

9-3.15A Wood Joists

Joists must be installed per the authorized shop drawings. The joist spacing, and span length must not be exceeded what is shown.

Inspect the joists for damage. Used joists should be examined for evidence of damage, decay, or distortion of shape, and defective or substandard caps rejected.

Measure the joists to determine their actual dimensions. If the actual dimension is smaller than the dimension assumed in the design, the joists may not be capable of carrying the intended load without overstress.

9-3.15B Steel Joists

Joists must be installed per the authorized shop drawings and be used in compliance with the manufacturer's instructions. The joist spacing and span length must not exceed what is shown.

Inspect the joists for damage. Examine used joists for distortion, bent flanges or webs and holes in flanges or webs. Defective or substandard joists must be rejected.

Measure the joists to verify the actual shape and size. If the actual shape and size is different than what is shown on the shop drawings, the joists may not be capable of carrying the intended load without overstress.

9-3.16 Soffit Plywood

Soffit plywood sheets should be set with the face grain perpendicular to the joists with abutting ends of the sheets supported on a common joist unless the design warrants otherwise. Refer to *Standard Specifications,* Section 51-1.03C(2), *Concrete Structures – Preparation – Forms,* for requirements for installation of soffit plywood.

9-3.17 Lost Deck Forms

Lost deck forms must be installed per the authorized shop drawings. Verify the forms are installed to the correct grade. Refer to *Standard Specifications,* Section 51-1.03C(2), *Concrete Structures – Preparation – Forms,* for requirements for installation of lost deck forms.

9-3.17A Driven Nail Anchorages

Standard Specifications, Section 51-1.03C(2)(a), *Concrete Structures – Preparation – Forms – General,* permits the use of driven type anchorages to fasten forms to interior surfaces of girder stems in box girder bridges where:

- Girders have more than 2-inch cover over the reinforcement.
- Anchorages do not penetrate the girder more than 2 inches.
- Anchorages have a minimum spacing of 6 inches.
- Anchorages are placed at least 3 inches clear from the edge of concrete.

9-3.17A(1) Inspection

The specification allows the use of nails driven with low velocity powder actuated hammers provided they are installed in conformance with the following criteria:

- Per the manufacture's recommendations
- Girders have more than 2 inches of cover over the reinforcement

- Anchorages do not penetrate the girder more than 2 inches and have a minimum spacing of 6 inches.
- Anchorages are placed at least 3 inches clear from the edge of concrete.
- Minimum end and edge distances for wood members must not be less than that required in Section 5-3.03, *Nails and Spikes*.

9-3.18 Deck Overhang

Deck overhangs are typically either supported by falsework resting on stringers or supported by overhang brackets.

Overhang falsework must be installed per the authorized shop drawings. Inspect the members for damage. Used members should be examined for evidence of damage, decay, or distortion of shape, and defective or substandard members rejected.

Measure the members to determine their actual dimensions. If the actual dimension is smaller than the dimension assumed in the design, the members may not be capable of carrying the intended load without overstress.

9-3.18A Deck Overhang Brackets

Refer to Section 7-3.04, *Deck Overhang Brackets,* and Figure 7-5, *Deck Overhang Brackets*. Several types of commercial and noncommercial metal brackets specifically designed to support cantilevered deck overhangs are available. On some brackets the diagonal leg is wood. On precast concrete (PC) girders, these brackets are typically supported by beam hangers or by form bolt inserts cast into the top of the PC girder stems. On steel girders these brackets are typically supported by threaded rods or bolts extending through holes drilled in the web of steel girders. The brackets typically have a diagonal leg braced against the bottom flange of the girder.

Verify that the overhang brackets are installed per the authorized shop drawings and used in compliance with the manufacturer’s instructions. Verify that the loads are applied at the intended location and that screw jacks are within the allowable range.

9-3.19 Tell-Tales

The *Standard Specifications,* Section 48-2.03C, *Falsework – Construction – Erection,* requires the contractor to use tell-tales to measure the settlement during concrete pours. Tell-tales should be attached to soffit forms or joists next to stringers and located as close as possible to the supporting bent cap or post. Enough tell-tales must be provided to determine the total settlement where concrete is being placed. The tell-tale must be readable from the ground. Typically, the tip of the tell-tale hangs free next to a stake in the ground. The initial reading of the tell-tale is marked on the stake before the concrete pour. Stakes must be placed outside of the pad area, so they are not affected by pad settlement. The tell-tale can also be referenced to a stationary point marked on a permanent structure, such as a nearby bridge column, for added monitoring. The movement of the tell-tale must be inspected regularly during the pour. If the tell-tale movement exceeds the maximum allowable per the *Standard Specifications,* Section 48-2.03C, *Falsework – Construction – Erection*, the pour must be stopped, and the contactor must apply corrective measures.

Tell-tales used to depict beam deflection should be placed at the required locations.

9-3.20 Falsework Over or Adjacent to Roadways or Railroads

9-3.20A Introduction

Refer to Section 4-12, *Falsework Over or Adjacent to Roadways and Railroads*, for special requirements that apply to falsework over or adjacent to roadways and railroads that are open to traffic. These requirements are included to provide higher standards of design and construction at locations where public safety is involved.

9-3.20B Stability

The *Standard Specifications,* Section 48-2.03C, *Falsework – Construction – Erection*, require that if falsework is over or adjacent to roadways or railroads, all details of the system that contribute to horizontal stability and resistance to impact, except for bolts in bracing, must:

- Be installed when each element of the falsework is erected.
- Remain in place until the falsework is removed.

For administration of this requirement, the following is provided:

- The requirement applies to the connections that provide lateral restraint at the base of the falsework post, at the top of the post between the post and cap, and between cap and stringer.
- The requirement applies to permanent bracing, which contributes to horizontal stability. Connections on wood bracings may be nailed rather than bolted to facilitate adjustment of the falsework bent to grade. Nailed connections, when used in lieu of bolts, must provide the same capacity as the permanent bolted connection. The permanent bolted connection must be installed within 48 hours after the completion of falsework grade adjustment.
- If traffic is being detoured during falsework erection, the components covered by the specification need not be installed as the falsework is erected but must be installed before traffic is allowed to pass adjacent to or under the falsework.

9-3.20C Horizontal and Vertical Clearances

Refer to Section 4-12.02, *Falsework Openings*, for information on submittal requirements for horizontal and vertical clearances.

For horizontal clearance refer to Section 4-12.03, *Horizontal Clearance*, Table 4-1, *Clearance to Railing Members and Barriers*, Figure 4-10, *Clearance to Railing Members and Barriers,* and *Standard Specifications,* Section 48-2.02B(4), *Falsework – Design Criteria – Special Locations*.

Horizontal and vertical clearances must be measured to verify compliance with contract requirements as soon as the bents are erected and the stringers set in place. Actual clearances should be recorded in the job records. The actual vertical clearance provided when the falsework is first erected must include an allowance for beam deflection and settlement that will occur as the concrete is placed. Any deviations from the original submitted clearances must be reported immediately, see Section 4-12.02, *Falsework Openings*.

Do not allow any falsework erection if the actual clearance is less than reported. Report new anticipated vertical clearance restrictions immediately. Falsework erection may commence 15 days after the resident engineer has submitted the new clearances. See Section 4-12.02, *Falsework Openings*.

9-3.20D Post Material and Parameters

Falsework posts must be either:

- Steel with a minimum section modulus of 9.5 in^3 about each axis.
- Sound timber with a minimum section modulus of 250 in^3 about each axis.

When pipe frame or tubular steel components are used in falsework over or adjacent to a roadway or railroad, either as individual posts or as legs in a tower bent, the specified minimum section modulus for steel columns will apply to the post or tower leg, but not to the screw jack extension.

9-3.20E Impact Loads and Mechanical Connections

Refer to Section 4-12.05C, *Impact Loads and Mechanical Connections,* for design considerations. The purpose of these mechanical connections is to resist lateral impact loads from traffic and should be oriented accordingly. Mechanical connections must be installed per the authorized shop drawings and are required at these locations:

- Bottom of posts: The requirements below apply to all posts within the limits shown in Section 4-12.01, *Introduction*, but not less than two posts. Lateral restraints must be effective parallel to and perpendicular to the bent. For a bent in a highway median, restraint must be effective in all four directions:

- Bottom of posts on corbels: Each post must be mechanically connected to its supporting footing or otherwise laterally restrained.
- Bottom of posts on bottom caps: Each post must be mechanically connected to the bottom cap. The bottom cap must be mechanically connected to its supporting footing or otherwise laterally restrained. The bottom cap must have a minimum of two restraints in the perpendicular direction, one near each end, and a minimum of one restraint in the longitudinal direction at one end.

- Top of post: The requirements below apply to all posts within the limits shown in Section 4-12.01, *Introduction*, but not less than two posts. Lateral restraint must be effective in all directions:
  - Each post must be mechanically connected to the top cap or stringer.
- Stringers over roadway: Mechanically connect these stringers to cap or framing:
  - Exterior stringers
  - Stringers adjacent to ends of discontinuous caps
  - Stringers over point of minimum vertical clearance
  - Every 5th stringer
- Stringers adjacent to roadway within the limits shown in Section 4-12.01, *Introduction*:
  - Mechanically connect exterior stringer adjacent to the roadway to cap or framing
- Stringers over railroad:
  - Mechanically connect all stringers to cap
- Stringers adjacent to railroad within the limits shown in Section 4-12.01, *Introduction*:
  - Mechanically connect all stringers within the limits to cap
- Timber bracing within the limits shown in Section 4-12.01, *Introduction*:
  - Bent parallel to roadway or railroad: Bolted connection required on all braces
  - Bent at an angle to roadway: Bolted connection required on all braces within the limits shown in Section 4-12.01, *Introduction*, and at least one brace bolted
  - Bent at an angle to roadway: Bolted connection required on all braces

9-3.20F Lighting at Traffic Openings

The lighting plan may be a separate action submittal or be part of the shop drawings.

Standard Specifications, Section 48-2.01D(3)(b), *Quality Assurance – Falsework Lighting*, Section 48-2.02C, *Materials – Falsework Lighting*, and Section 48-2.03E, *Construction – Falsework Lighting,* state the requirements for pavement and portal lighting at traffic openings. Any project specific requirements will be shown on the plans or included in the special provisions.

Falsework lighting must be installed per the authorized lighting plan. All features of the portal illumination, including plywood clearance markers, as well as pavement and pedestrian walkway lighting, if required, must be in place and operational:

- The same night the first post adjacent to roadway, railroad, or pedestrian walkway is erected.
- Immediately after falsework bents are erected adjacent to roadway, railroad, or pedestrian walkways.
- Prior to allowing traffic adjacent to the falsework.

As soon as the falsework is erected, and the lights turned on, the lit falsework opening should be inspected after dark to check the effectiveness of the lighting, and the lights moved or adjusted if necessary to provide uniform illumination. Lighting fixtures must be aimed to avoid glare to oncoming traffic. Inspection at night should continue periodically, as lights may be inadvertently moved or disturbed as construction continues. An inspection during adverse weather, such as rain or fog, is also advisable.

Temporary K-rail and all painted surfaces at the portal opening must be maintained in a clean, white condition. Repainting may be necessary on occasion. Refer to the contract documents for payment of such work.

The *Standard Specifications,* Section 48-2.03E(1), *Falsework Lighting – General,* do not permit closing of traffic lanes for routine maintenance of the lighting system on any roadway having a posted speed limit above 25 miles per hour.

9-3.21 Cal-OSHA Requirements

The Cal-OSHA Construction Safety Orders require the contractor to obtain a permit to construct or remove falsework or shoring that is more than three stories high. This requirement is discussed in Chapter 2, *Review of Shop Drawings*.

The Cal-OSHA Construction Safety Orders §1717, *Falsework and Vertical Shoring,* requires all falsework or vertical shoring systems to be inspected and certified prior to concrete placement. The certification must be in writing, available on site, and must state that the falsework, as constructed, substantially conforms to the working drawings and that the materials and workmanship are satisfactory for the purpose intended.

For falsework or shoring which exceeds 14 feet in height, measured from the top of the lower cap to the superstructure soffit, or where the length of an individual span exceeds 16 feet, or where provision is made for the passage of vehicular or railroad traffic through the falsework or shoring, the required inspection and certification must be made by a civil engineer registered in the State of California, or by his authorized representative.

For all other falsework, the inspection and certification may be made by any one of the following:

- A civil engineer registered in the State of California
- For shoring systems, a manufacturer's authorized representative
- A licensed contractor's representative qualified in the usage and erection of falsework and vertical shoring

Arranging for the required inspection and certification is the contractor's responsibility. When the falsework design is such as to require inspection and certification by a registered civil engineer, it is the contractor's engineer who assumes this responsibility.

Inspection and certification of the falsework pursuant to the requirements in Article 1717 of the Construction Safety Orders does not relieve the contractor of any of his responsibilities under the contract for falsework construction, nor does it relieve the structure representative of his responsibilities with respect to contract administration. Even though the falsework is certified by the contractor's engineer or by other appropriate authority, the structure representative must satisfy himself that the falsework has been constructed in conformance with the authorized shop drawings before permitting the contractor to place concrete.

9-3.22 Field Changes

The falsework must be constructed per the authorized shop drawings. Per the *Standard Specifications,* Section 5-1.23B(2), *Shop Drawings*, any changes require that the revised drawings be submitted for review in the same manner as the original drawings.

In some cases, the change may be small and can be shown on a simple sketch; however, the sketch must be signed and stamped by a civil engineer registered in the State of California. Calculations are required in all cases.

Contractually, the review time for resubmittal due to field changes is the same as for the original submittal; however, it is SC practice to give high review priority to resubmittals during falsework construction.

Work shown on a revised shop drawing or sketch may not begin until that drawing has been authorized.

9-3.23 Adjustments

Falsework adjustment includes any adjustment or grading.

Particular attention should be given to falsework bents where adjustment is provided at the bottom of the posts. Since any differential vertical movement of posts within a bent may induce undesirable stresses into the frame, the diagonal bracing should be inspected after the falsework is adjusted for evidence of deflected braces and/or distortion at the connections.

Jacks used for grading or adjusting the falsework must be plumb and not extended beyond the distance recommended by the jack manufacturer. The load should be centered over the jack e.g. cap beam web should line up with the center of the jack piston.

Proper bracing must be in place during jacking operations, including additional temporary bracing as required by the shop drawings. Release of falsework bracing must follow the authorized procedure shown in the falsework submittal. When cable bracing is being adjusted, devices such as pork-chops and come-alongs must be used to control the tensioning and de-tensioning of the cables. Releasing cable clips without controlling the tension in the cable is not acceptable.

9-3.24 Metal Shoring Systems

9-3.24A Introduction

This chapter describes the general inspection procedure for metal shoring systems. The term "metal shoring system" describes falsework consisting of individual components that may be assembled and erected in place to form a series of internally braced metal towers of any desired height. The tower legs, directly, or through a cap system, support the main load carrying members and transmit the load to a stable foundation.

Refer to Section 7-4, *Metal Shoring Systems,* for additional information.

9-3.24B Inspection

The safe load carrying capacity of all commercially available shoring systems is based on the use of new components, or used components in good condition, properly erected in conformance with the manufacturer's recommendations. Therefore, proper inspection is of particular importance to verify the adequacy of the completed system. Following is a list of items to be inspected and things to consider:

- The shoring must be installed per the authorized shop drawings and per the manufacturer’s recommendations.
- Shoring components should be inspected prior to erection.
- Any component that is heavily rusted, bent, dented, or otherwise defective, should be rejected.
- Any fabricated unit in which individual members are bent, dented, twisted or broken, or where the welded connections are cracked or shows evidence of re-welding, should be rejected.
- Shoring towers must be installed plumb on level foundations. Shoring must be plumb in both directions. Refer to technical data sheets issued by the manufacturer for the maximum allowable deviation from true vertical. If this deviation is exceeded, the shoring must be readjusted to meet the limit.
- A base plate or shore head device should be used at the top and base of all tower legs. All base plates and shore heads must be in firm contact with the footing at the base and the cap at the top.
- Screw jack extension device may be used at the top of all tower legs. All extension devices must be in firm contact with the cap at the top. Screw jacks will not be allowed at the bottom of shoring towers.
- Shore heads and extension devices must be axially loaded, since shoring components are not designed to resist eccentric loads.
- Vertical components should fit together evenly, without any gap between the upper end of one unit and the lower end of the other unit.
- Base plates, shore heads, and screw jack extension devices must fit into the tower legs.
- Any component that cannot be brought into proper contact with the component into or onto which it is intended to fit should not be used.
- All locking devices on frames and braces must be in good working order. Coupling pins must bring the frame or panel legs into proper alignment and pivoted cross braces must have the center pivot in place.
- Shoring components should be identifiable by paste on stickers or by alpha-numeric stamped impressions.
- Commercial shoring systems used for falsework are reviewed and authorized based on a particular system from specific manufacturers. Various systems may have many similar components, but they are not intended to be interchangeable

between systems. Field engineers should verify that the system furnished is the system shown on the shop drawings and further, that all system components are part of the authorized system. Intermixing of components of various systems is not acceptable.

9-3.24C Letters of Certification

These certifications are required for falsework construction:

- The *Standard Specifications,* Section 48-2.01C(1), *Falsework – Submittals – General,* requires the contractor to submit a letter of certification, which certifies that all components of the manufactured assembly are used in accordance with the manufacturer's instructions.
- In addition, the *Standard Specifications,* Section 48-2.01C(2), *Falsework – Submittals – Shop Drawings,* and the Cal-OSHA Construction Safety Order §1717(c)(1), *Falsework and Vertical Shoring – Inspection,* require another certification, which certifies that the falsework (which the shoring is part of) is constructed as shown on the authorized shop drawings before concrete is placed.

9-3.25 Manufactured Assemblies

The term *manufactured assembly* means any commercial product, the use of which is governed by conditions and/or restrictions imposed by the manufacturer. Manufactured assemblies routinely used in falsework construction include products such as jacks, hangers, clips, brackets, and similar hardware products, as well as all types of manufactured shoring systems. When authorized for use, such products may be incorporated into the design.

The *Standard Specifications,* Section 48-2.01C(1), *Falsework – Submittals – General,* require the contractor to furnish a written certification stating that all components of the assembly are used in accordance with the manufacturer's recommendations.

When a manufactured assembly is used in the falsework, the contractor must furnish a written certification stating that all components of the assembly are used in accordance with the manufacturer's recommendations. A separate certification is required for each product or device used in the falsework. See Section 9-3.27B, *Manufactured Assemblies*.

9-3.26 Falsework Certification

9-3.26A Falsework

The *Standard* Specifications*, Section* 48-2.01C(2), *Falsework – Submittals – Shop Drawings,* require that the civil engineer registered in the State of California who signs the shop drawings must certify that the falsework is constructed as shown on the shop drawings before concrete is placed. This certification includes all falsework components, such as soffit forms, stringers, caps, post, pads, bracing, connections and all manufactured assemblies, such as overhang brackets and shoring towers. The certification must be in writing, and it must state that the falsework, as constructed, conforms to the shop drawings and that the materials and workmanship are satisfactory for the purpose intended.

Arranging for the required inspection and certification is the contractor's responsibility. It is the engineer who signed the shop drawings who assumes this responsibility.

9-3.26B Manufactured Assemblies

The *Standard Specifications,* Section 48-2.01C(1), *Falsework – Submittals – General,* requires the contractor to submit a letter of certification, which certifies that all components of the manufactured assembly are used in accordance with the manufacturer's instructions. This certification is in addition to the certification by the civil engineer registered in the State of California in Section 9-3.26A, *Falsework*.

Arranging for the required inspection and certification is the contractor's responsibility. It is the contractor’s engineer or a representative from the manufacturer who assumes this responsibility. However, the structure representative must verify that the manufactured assemblies have been inspected by examining the certificate and noting its existence in the project diary. A copy of the certificate must be placed in the job records.

9-3.27 Bridge Construction Engineer Review

The Bridge Construction Engineer must perform a field review of the installations, together with the structure representative, before concrete is placed. This review typically takes place after the contractor has certified the falsework.

9-4 Concrete Placement

9-4.01 Inspection During Concrete Placement

As concrete is being placed, the falsework should be inspected at frequent intervals. In particular, look for the following indicators of incipient failure:

- Excessive compression at the tops and bottoms of posts and under the ends of stringers. Crushing of wedges. Settlement of sand jacks.
- Movement or deflection of diagonal bracing. Distortion at connections. Pulling of nails.
- Tilting or rotation of joists or stringers. Excessive deflection of any horizontal member.

- Posts or towers that are bowing or moving out of plumb.
- Excessive settlement as indicated by tell-tales.
- The sound of falling concrete or breaking timbers. Any unusual sound.

The *Standard Specifications,* Section 48-2.03C, *Falsework – Construction – Erection,* limit falsework settlement to a maximum of ±3/8-inch deviation from the anticipated settlement shown on the shop drawings. The movement of the tell-tale must be inspected regularly during the pour. If the tell-tale movement exceeds the maximum allowable, the pour must be stopped, and the contractor must apply corrective measures. Concrete placement must not be resumed until the engineer is satisfied that further settlement will not occur. Settlement due to soil compression may continue for some length of time, even though the load is not increased.

If inspection reveals members in distress such as crushing at joints, rotation or tilting of vertical members, or any similar indication of incipient failure, all concrete placement must be stopped immediately, and the falsework strengthened by the installation of supplementary supports, or by some other means. Refer to the SC *Concrete Technology Manual* for a discussion of the factors to be considered when it becomes necessary to install an emergency construction joint.

For continuous steel or precast concrete girders the Standard Specification 51-1.03D(2), *Concrete Bridge Decks,* requires the portion of deck over the supports to be placed last. Verify that the deck placement plan shown on the project plan or on the authorized shop drawings is implemented by the contractor, including the location of all the construction joints.

Section 2-4.01, *Initial Review,* requires that if a concrete placing schedule is not shown on the contract plans, the shop drawings must include a superstructure placing diagram showing the proposed concrete placing sequence and/or the direction of pour, whichever one is applicable, and location of all construction joints.

If falsework bents are located near existing roadways or railroads it is likely that the soil in the area is stiffer and the bents in this area will pick up more load. Although these bents have been designed for an increase in load, it is prudent to pay close attention to these bents during the pour.

9-4.02 Inspection After Concrete Placement

Falsework inspection should not stop with concrete placement but should continue periodically until the falsework has been completely removed.

One important, and often overlooked point, is the danger of rain and curing water softening the falsework foundation. Some means should be provided to prevent curing water from reaching and soaking the foundation material beneath the falsework pads.

The contractor should provide for drainage of rain or curing water that accumulates in the box girder cells. Such water in the cells could easily overstress the falsework or, if deep enough, the permanent structure as well. If the water is allowed to drain through the soffit drain locations, ensure the path of drainage will not adversely affect the system. The contractor must comply with the requirements in the authorized Water Quality Control Plan (WQCP) and Storm Water and Water Pollution Prevention Plan (SWPPP) when curing concrete with water.

9-4.03 Deck Shrinkage

Continuing inspection is particularly important in the case of post-tensioned structures because of the redistribution of dead load forces that occurs following the deck concrete pour. As the newly placed deck concrete shrinks during the curing period, the downward force exerted on the falsework by the bridge superstructure can be re-distributed. The increase is greatest near the center of the structure span, and typically reaches its maximum from four to seven days after the deck concrete is placed.

The effect of deck shrinkage is of greater concern in cast-in place prestressed structures than in conventionally reinforced concrete structures because post-tensioned structures have relatively little rigidity until they are stressed.

The effect of deck shrinkage is not addressed in the specifications; however, field engineers should be aware of the potential shrinkage and look for locations where the falsework may be adversely affected.

9-5 Removal

9-5.01 Introduction

The *Standard Specifications,* Section 48-2.03A, *Falsework – Construction – General,* require the use of construction methods, which include temporary bracing when necessary, to withstand all imposed loads and to prevent overturning or collapse of the falsework during erection, construction, and removal. The means by which the contractor complies with this specification requirement is commonly referred to as the erection plan and removal plan.

Before falsework removal begins, the removal plan, as described in the authorized shop drawings, should be discussed with the contractor's field representative who will be responsible for supervising the removal. The purpose of this requirement is to verify that the removal plan is appropriate for the site and conditions to be encountered, and that all persons involved with the work, both the contractor and State, are familiar with the removal plan.

Standard Specifications, Section 48-2.03D, *Falsework – Removal,* discusses falsework removal. Removal is not allowed over traffic per *Standard Specifications,* Section 48-2.03A, *Falsework – Construction – General*.

In general, the falsework must remain in place for a specified time period, or until the concrete attains a specified strength, or for cast in-place prestressed construction, until stressing (but not grouting) is completed.

For continuous structures, the removal of falsework supporting a given span cannot begin until all required work, excluding concrete above the bridge deck and grouting of prestressing ducts, has been completed in that span and in the adjacent spans over a length equal to at least l/2 of the length of the span where falsework is to be removed. See Figure 9-8, *Falsework Removal of Continuous Bridge*. The reason for the restriction is that in continuous concrete bridges the bending moment in the finished bridge is reduced due to adjacent spans acting like counterweights. Therefore, placing half of the adjacent span concrete, **L/2**, will keep the positive and negative moments in span A close to design dead load moments.

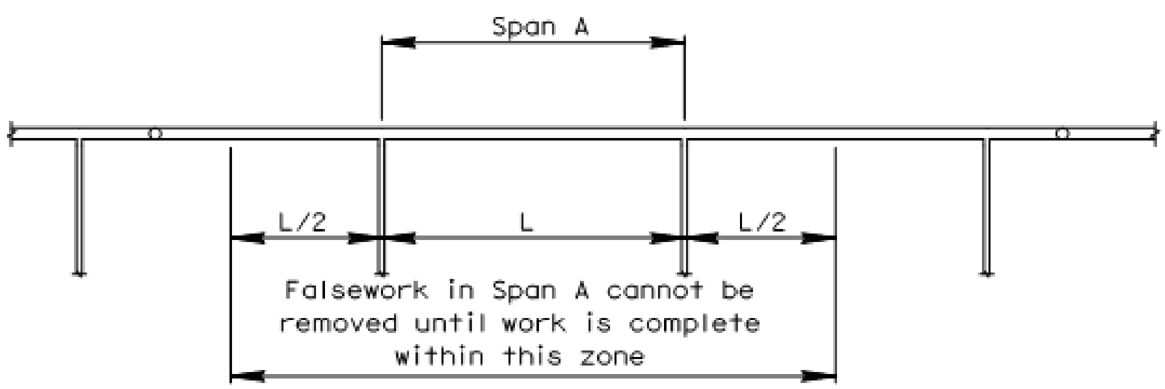


Figure 9-8. Falsework Removal of Continuous Bridges.

Removing falsework in the span adjacent to the short side of the hinge is not allowed until all work at the hinge is complete. Removal of falsework in the span adjacent to the short side of the hinge would cause rotation of the hinge and induce excessive dead load moments in the bridge elements. Proposals to provide alternative loads at the hinge or alternative support to the adjacent span to allow removal of the falsework in the span adjacent to the short side of the hinge should be discussed with the bridge designer.

9-5.02 Removal Procedure

Falsework removal often presents a greater challenge than erection because the new bridge reduces the available space, hence it must be carefully performed to ensure both worker and public safety. The *Standard Specifications,* Section 48-2.01C(2), *Falsework – Submittals – Shop Drawings* , require the contractor to include a removal plan on the shop drawings. The plan must show the methods and procedures to be employed and any temporary bracing required.

The stability of the system depends on the interaction of many component parts. As falsework components are removed, unbalanced and/or eccentric loadings may occur, and the use of jacks to unload portions of the falsework may induce forces that exceed those considered in the original design. No stabilizing component should be removed without considering the effect of its removal on the stability of the falsework still in place.

Since the falsework removal plan must be shown on the shop drawings, it is necessary for the falsework designer to develop the falsework removal plan many months (even years on very large projects) before the actual removal will take place, and thus its appropriateness may be affected by conditions and circumstances that were not anticipated. In view of this reality, prior to the start of any removal, the structure representative and field engineers are to meet with the contractor to review the removal plan. The review should consider the general appropriateness of the removal method in the light of the actual site conditions and should include a discussion of the removal sequence and equipment to be used, the number and responsibilities of the workers involved, and public and worker safety.

The contractor must designate an employee who will be in charge of the falsework removal work and who will be present at the work site while the work is in progress. Additionally, the structure representative must assign a field engineer to be present whenever falsework is being removed. However, falsework removal, like all other contract work, is the contractor's operation, and it is the contractor's responsibility to perform the work in a safe manner and in accordance with the authorized removal plan.

Some contractors use cables attached to winches set on the bridge deck to lower elements of the falsework system. While this is a simple and generally satisfactory removal method, the weight of the winch plus the weight of the suspended falsework may produce a relatively large concentrated load. Before such removal plans are authorized, the structure representative should be certain that the winch load is distributed over the deck in a manner that prevents overstressing of the permanent structure. In some cases, it may be appropriate to discuss the distribution method with the bridge designer.

All bracing must remain in place until the falsework is removed per the removal plans on the authorized shop drawings. If a contractor wants to remove some bracing or other portions of the falsework during the curing period prior to the bridge being self-supported the procedure must be clearly shown on the authorized shop drawings.

9-5.02A Removal with Winches or Similar Systems

The *Standard Specifications,* Section 48-2.03D, *Falsework – Removal*, requires falsework removal systems employing methods of supporting falsework by winches, hydraulic jacks with prestressing steel, HS rods, or cranes must be supported by an independent support system when the falsework is over vehicular, pedestrian, or railroad traffic. When traffic is detoured for lowering falsework the independent support system is not required.
Falsework must never be lowered over live traffic, see *Standard Specifications,* Section 7-1.04, *Public Safety*. The remaining falsework must be inspected before opening to traffic.

High strength rods are typically used as an independent system. All falsework must be tight against the soffit and no loose soffit material suspended over the traffic or construction.

The following are some restrictions on the location and use of winches:

- Winches installed on the deck must be plumb within 2% or as noted on the authorized plans. This can be achieved by installing full length shims at a minimum of three locations under the winch frame beams. The shims can result in higher concentrated loads and the contractor must verify that the load will not damage the deck.
- Restriction on winch orientation with respect to bridge centerline is as follows:
    - When the winch nose is over the edge of deck then the skew in plan view is very restrictive.
    - When the winch nose is at the bridge overhang then rear winch support must be on the exterior girder.
    - When the winch nose is over the bay then the winch can be placed at any angle except that the rear winch support must not be placed on the bridge overhang.
- The rod and wire rope must have a softener at the winch cable hole.

9-5.03 Stage Construction

When continuous cast-in-place prestressed structures are constructed in stages, the stage construction sequence will require some load supporting elements of the falsework system to remain in place for an extended period of time. For such structures, falsework removal involves special considerations.

For any given construction stage, the initial stressing will transfer the superstructure dead load from the center of the spans toward the points of support. This redistribution of dead load forces will decrease the load applied to the falsework near the center of the continuous spans. The load being carried by falsework near the center of a suspended span will be decreased as well; however, the load on the falsework supporting the hinge or construction joint will be increased by dead load transfer.

For continuous prestressed structures, the specified sequence of falsework removal will require certain elements of the system to remain in place. All components of such falsework, including diagonal bracing, must remain in place, even though the falsework may have been partially unloaded by the prestressing operation. This procedure is necessary because, with the passage of time, the redistributed dead load will be carried back toward the center of the span as superstructure dead load deflection takes place.

For continuous prestressed structures, all elements of the falsework system that are not required by the specifications to remain in place should be completely removed. If the falsework cannot be removed within a reasonable time, any components remaining in place should be unloaded. This procedure is necessary to prevent overloading of partly disassembled falsework still in place under the deflecting superstructure.

Appendix A: Wood Characteristics

Table of Contents

A-1 Introduction

This appendix discusses the characteristics of natural sawn lumber. Sawn lumber is commonly used in falsework construction. This appendix discusses the structure of wood, strength of wood, and defects in wood.

A-2 Structure of Wood

A-2.01 Wood Cells

The cells making up the structural elements of wood are generally tubular and quite firmly grown together. Dry wood cells may be empty, or partly filled with deposits such as gums and resins.

Many wood cells are considerably elongated and pointed at the ends. Such cells are called fibers. The direction of the wood fibers with respect to the axis of the tree is one of the most important characteristics affecting the usefulness of a given piece of wood, since it has a marked influence on strength.

The length of wood fibers may vary considerably in a given tree, as well as between different species. Typically, hardwood fibers average about 1/25 inch in length and softwood fibers average from 1/8 to 1/4 inch in length, or longer.

A-2.02 Growth Rings

Between the bark of a tree and the wood interior is a layer of thin-walled, nearly invisible living cells, called the cambium layer, in which all growth of the tree takes place. New wood cells are formed on the inside and new bark on the outside of the cambium. No growth in either thickness or length takes place in wood already formed, new growth is purely the addition of new cells, not the further development of old ones.

In temperate climates there is usually enough difference in color and texture between the wood formed early and that formed late in the growing season to produce well marked annual growth rings. The age of a tree at any cross section may be determined by counting the growth rings. One ring represents one year of growth, provided the growth has been interrupted by cold or dry seasons so that the change in cell structure is sufficient to define the annual layer.

A-2.03 Springwood and Summerwood

In many species of wood, each annual growth ring is divided into two distinct layers. The inner part of the ring, formed first in the growing season, is called springwood. The outer part, formed later in the growing season, is called summerwood. The transition from springwood to summerwood may be gradual or abrupt, depending on the kind of wood and the growing conditions when it was formed.

Springwood is generally characterized by cells with relatively large cavities and thin walls, whereas summerwood cells have smaller cavities and thicker walls. Summerwood generally will be heavier, harder and stronger than springwood.

The percentage of summerwood in a given piece of lumber determines the density of the piece. Other factors being equal in a piece of lumber, the higher the density, the greater the strength. Because of its greater density, the proportion of summerwood in a particular piece of lumber is sometimes used as an indication of its quality and strength.

A-2.04 Sapwood and Heartwood

The wood portion of a tree has two main parts. The outer part, which consists of a ring of wood around the tree just under the bark, called sapwood. Within the sapwood ring is an inner core, generally darker in color, called heartwood.

The sapwood ring varies in thickness from one to three inches, depending on the age and species of the tree. Sapwood contains the living cells and takes part in the active life processes of the tree. Heartwood consists of inactive (not dead) cells and serves mainly to give strength to the tree. Except for the slightly darker color of heartwood, there is little difference in the strength or physical characteristics of heartwood and sapwood from a given tree.

As a tree grows older and larger, the inner layers of sapwood change to heartwood. Eventually the heartwood core forms the major part of the trunk and main branches.

A-2.05 Chemical Composition of Wood

Wood is a complex aggregate of compounds, which may be divided into two major groups:

1. Compounds making up the cell structure
2. All other substances, which are commonly called “extractives” or infiltrated materials

The cell wall components consist primarily of cellulose and lignin. Cellulose is the most abundant constituent, comprising about 70 to 80% of the wood structure. Lignin, which comprises from 20 to 30% of the wood structure, is the cementing agent which binds the individual wood fibers together to form a substance of strength and rigidity.

In addition to cellulose and lignin, wood contains a small amount of mineral matter. These minerals, known as “ash forming” minerals because they are left as ash when the lignin and cellulose are burned, constitute less than one percent of the total wood substance.

The extractives are not part of the wood structure as such, but they contribute such properties as color, odor, taste and resistance to decay. They include tannins, starches, oils, resins, acids, fats and waxes. They are found within the hollow portions of the wood cells and can be removed from the wood by neutral solvents such as water, alcohol, benzol, acetone, and ether.

A-2.06 Hardwoods and Softwoods

All wood species are classified for commercial purposes as either hardwoods or softwoods.

Hardwoods are broad-leafed, deciduous trees which drop their leaves at the end of the growing season. Examples of commercially grown hardwood trees include oak, maple, walnut, and ash.

Softwoods are evergreen trees. Evergreen trees may have fern-like leaves, typical of the redwoods, or needle-shaped leaves typical of the pines and firs. Softwoods are also known as conifers (or “cone bearers”) because all softwood trees bear cones of one kind or another.

The terms hardwood and softwood are somewhat misleading in that they have no direct application to the actual or relative hardness or softness of a particular kind of wood. Many hardwoods are softer than the average softwood. Douglas Fir, which is widely used in the west as a construction material, is a softwood by definition; nevertheless, the better grades of Douglas Fir are dense, hard, and tough.

A-2.07 Specific Gravity

Although dry wood of most species will float in water, the absolute specific gravity of the basic substance of which wood is composed is about 1.55 for all species. Thus, it is evident that cell cavities and pores occupy a large part of the volume of wood, so that the resultant relative specific gravity of wood is less than 1.00 for most species.

Variation in the size of the cell openings and the thickness of the cell walls causes some species to have more wood substance than others; and therefore, to have higher relative specific gravities. Therefore, the density of cut lumber will vary between species, averaging from 30 to 40 pcf at normal moisture content for most commercially grown softwoods. Since density depends on the amount of wood substance in a given piece of lumber, it is an excellent index of strength. The higher the density, the greater the strength of cut lumber, all other factors being equal.

A-2.08 Grain

The term grain as it is applied to wood is most often used to indicate the direction of the wood fibers relative to the axis of the tree or the longitudinal edges of a piece of cut lumber. Thus, if the fibers are generally parallel to the axis of a tree, the lumber from the tree will be straight grained; however, if the direction of the fibers makes an angle with the axis, the lumber will be cross grained. The term cross grain is also used to indicate a direction which is actually perpendicular to the grain. This usage is generally associated with the direction, with respect to the grain, at which a load is applied.

Grain is also used in reference to the width and spacing of the annual growth rings. Thus, lumber may be close grained, medium grained, or coarse grained. However, these are relative terms without precise meaning.

Edge grain refers to lumber in which the growth rings are at approximately right angles to the surface of the piece. Flat grain refers to lumber in which the surface of the piece of lumber is approximately tangent or parallel to the direction of the growth rings.

A-2.09 Moisture Content

Living trees may contain as much as 200% moisture by weight. After a tree is cut and converted into lumber, the wood begins to lose moisture. The process of removing moisture from green lumber is known as seasoning, which may be accomplished by exposure to the air or by kiln drying.

Green wood contains moisture in two forms, as free water in the cell cavities and as absorbed water in the capillaries of the cell walls. When green wood begins to lose water, the cell walls remain saturated until the free water has evaporated. The point at which evaporation of free water is complete and the cell walls begin to lose their moisture is called the fiber saturation point. The fiber saturation point occurs at a moisture content of about 25 to 30% for most species.

Variations in moisture content above the fiber saturation point have no effect on the volume or strength of wood. As wood dries below the fiber saturation point and begins to lose moisture from the cell walls, shrinkage begins, and strength increases.

Wood in use over a period of time will give off or take on moisture from the surrounding atmosphere until the moisture in the wood corresponds to the humidity of the surrounding atmosphere. When exposed to similar atmospheric conditions, different woods will have the same moisture content regardless of their density.

Moisture content has an important effect on susceptibility to decay. Most decay producing fungi require a moisture content above the fiber saturation point to survive. In addition, favorable temperatures, an adequate supply of air, and a source of food are essential. Wood that is continuously soaked in water (as when submerged) or is continuously dry (i.e., with a moisture content of 20% or less) will not decay.

A-2.10 Shrinkage

Shrinkage of wood takes place between the fiber saturation point and the oven dry condition. It is stated as a percentage of the original or green dimension.
Wood shrinkage is greatest in the direction of the annual growth rings, somewhat less across the rings, and very little along the grain. Typically, shrinkage along the grain (longitudinal shrinkage) is usually less than 1% and therefore too small to be of practical significance.

Shrinkage of commercial softwood boards across the grain averages about 1% for each 4% change in moisture content. Shrinkage of hardwoods is slightly larger.

Large structural timbers shrink proportionally less than smaller pieces of lumber because drying does not take place simultaneously in the inner and outer portions. The inner portion dries more slowly than the outer portion and prevents the wood near the surface from shrinking normally. Later, when drying of the interior occurs, the outer portion, which has now set, prevents the inner portion from shrinking to the extent that it otherwise would.

A-3 Strength of Wood

A-3.01 Introduction

The term strength as it is used in structural design terminology refers to the ability of a given material to resist elastic deformation when subjected to external forces. Unlike most other building materials, however, wood exhibits different strength properties depending on whether the forces are applied parallel or perpendicular to the direction of the wood fibers or grain of the wood. In general wood is strongest along the grain and weakest at right angles to it.

Because the strength of a given piece of wood depends on the direction of the wood fibers with respect to the direction of the applied load, it is necessary to consider the effect on wood strength of each of the stresses produced by a particular loading condition. These stresses, and the ability of wood to withstand them, are discussed in the following sections.

A-3.02 Tensile Strength

The tensile strength of wood parallel to the grain depends on the strength of the fibers and is affected not only by the nature and dimensions of the wood elements, but also by their arrangement. It is greatest in straight grained specimens with thick walled fibers.

Cross grain of any kind will materially reduce the tensile strength of wood, since tensile strength perpendicular to the grain is only a small fraction of the strength parallel to the grain. The ratio of tensile strength parallel to the grain to tensile strength perpendicular to the grain is commonly as high as 40 to 1. Therefore, cross grain tensile strength is assumed to be zero for all practical purposes and cross grain tension should be avoided.

When loaded in direct tension, strain and stress are proportional virtually to ultimate load, and there is no well-defined proportional limit below this point. Wood, therefore, will yield only a very slight amount prior to ultimate failure in direct tension.
As a matter of interest, if only the net cross sectional area of a piece of wood is considered (i.e., if the cell cavity area is deducted) the ultimate tensile strength of a clear specimen is about 70,000 psi, which is comparable to the strength of mild steel.

A-3.03 Compressive Strength

There are two ways in which wood may be subjected to compressive stress: compression perpendicular to the grain sometimes referred to as cross grain compression, and compression parallel to the grain.

Compression perpendicular to the grain is often critical in timber design. It is usually most severe at the ends of deep, narrow beams, and in the connecting members at the top and bottom of short, heavily loaded posts.

The primary effect of compression perpendicular to the grain is compaction of the wood fibers. As the fibers compact the load carrying capacity of the wood increases as the density of the material increases.

If the load is applied to only a portion of the upper surface, the bearing plate or post indents the wood, crushing the upper fibers without affecting the lower part of the member. Under this loading condition, the projecting ends of the member increases the strength of the material directly beneath the compressing weight by introducing a beam action, which helps support the load, however, this beam action is exerted for a short distance only.

Compression parallel to the grain will occur in many uses of wood (such as columns, props, and posts) in which the member is subjected to loads which tend to shorten it lengthwise.

The compressive strength of wood parallel to the grain is from three to five times greater than the compressive strength perpendicular to the grain. The ratio is about the same for both green and seasoned material.

Maximum compressive strength parallel to the grain is a measure of the ability of a short post to withstand load. In long posts, however, bending is introduced before the full crushing or compressive strength is reached, and failure is by lateral bending or flexure rather than by crushing.

In determining the strength of wood posts, the ratio of the unsupported length of the member to the least cross-sectional dimension is of primary importance. Short posts having an unsupported length of less than 11 times the least dimension have practically the full compressive strength of the material, whereas the strength of extremely long posts is governed entirely by the stiffness of the wood and resistance to endwise compression is not involved. For posts between these two extremes, both the compressive strength and the stiffness of the wood are taken into consideration.

A-3.04 Shear Strength

Shear strength is a measure of the ability of wood to resist forces that tend to cause one part of a member to slip or slide along another part adjacent to it.

Shear stresses will occur under almost all loading conditions, and the forces which produce them are classified according to the direction in which they act as shear parallel to (or along) the grain and shear perpendicular to (or across) the grain.

Under certain conditions, shearing stresses may act both perpendicular to the grain and parallel to the grain at the same time. For example, in a loaded beam the applied forces tend to shear the wood across the grain. This stress is equal to the resultant force acting perpendicular to the axis of the beam at any point. In a member uniformly loaded and supported at both ends, the stress is maximum at the points of support and zero at the center. In addition, there is a shearing force tending to move the fibers of the beam past each other in a longitudinal direction, or along the grain. In a beam this force is known as horizontal shear.

The presence of horizontal shear in the direction of the grain may be readily demonstrated by placing several boards, one on top of the other, and loading them at the center. As the boards bend, they slip over one another so that the ends of each project beyond those of the one below. In a solid beam this movement is restrained, and the maximum longitudinal shear stresses develop at the neutral plane and decrease toward the upper and lower edges.

The resistance of wood to shear perpendicular to the grain is much greater than its ability to withstand shear along the grain. So much so in fact, that shear perpendicular to the grain may be ignored in beams, stringers, and similar members. However, horizontal shear is frequently critical in beams and caps, particularly in the case of short, deep members, and should be considered when designing or checking any member which is subjected to bending stresses.

A-3.05 Flexural Strength

When external forces, acting in the same plane, are applied at right angles to the axis of a simple beam, causing it to deflect or bend, three fundamental stresses, tension, compression, and shear, will occur within the member, all acting in a direction parallel to the grain.

If the beam is loaded too heavily, it will break or fail in some manner. Beam failures are classified according to the way in which they develop, i.e. compression failure, tension failure, or horizontal shear failure. A combination of failures may develop if the beam is completely ruptured.

Since the compressive strength of wood parallel to the grain is normally greater than the tensile strength in the same direction, beam failure will occur first by tearing or rupture of the wood on the tension side followed by crushing on the compression side. Horizontal shear failure is fairly common when the ratio of the height to the span of the beam is relatively large.

A-3.06 Stiffness

The stiffness of wood, when used in reference to either a beam or long post, is a measure of its ability to resist deformation or bending. It is expressed in terms of the modulus of elasticity and applies only within the proportional limit.

Because of its fibrous structure, wood is characterized by three moduli of elasticity, one for each structural direction. Values for modulus of elasticity in the two directions perpendicular to the grain are relatively low, being approximately 1/12 to 1/20 of the value parallel to the grain. For solid wood beams, however, the value for modulus of elasticity parallel to the grain is the only one of importance.

The modulus of elasticity is used in calculating the deflection of beams and joists, and in computing safe loads for long and intermediate posts.

Although stiffness is independent of bending strength, woods which rank high in one respect usually rank high in the other as well.

A-3.07 Effect of Moisture Content

Wood increases in strength as it dries. The strength increase begins at the fiber saturation point (the point at which the cell walls begin to lose moisture) and increases rapidly as drying continues.

Drying wood from the fiber saturation point to 5% moisture will usually double, and in some cases triple the end crushing strength. However, the increase in bending strength is nominal. This increase in strength with seasoning is greater in small clear specimens of wood than in large timbers. In the latter, increase in strength may be offset to some extent by checking, if checking develops during the seasoning process.

Not all strength properties increase with a decrease in moisture content, in fact, properties indicative of toughness, or shock resistance may actually decrease as wood dries. This is because dried wood will not bend as far as green wood before failure (although it will sustain a greater load) and because toughness depends on both strength and flexibility.

A-3.08 Duration of Load

Wood has a unique property not found in other building materials. This is its ability to withstand a proportionally greater stress as the length of time the load is applied is decreased.

Both the elastic limit and the ultimate strength of wood are higher under short term loading than under long term loading. Wood is thus able to absorb overloads of considerable magnitude for short periods of time, or smaller overloads for longer periods of time, obviously, the duration of a load is an important factor in determining the total load that a member can safely carry.

A-4 Wood Defects

A-4.01 Definition

As defined by American Society for Testing and Materials (ASTM), a defect is any irregularity occurring in or on wood that reduces its strength as compared to the strength of a clear grained specimen.

A-4.02 Knots

A knot is that portion of a branch or limb which has been incorporated into the body of the tree. Knots are the most prevalent defect in structural timber.

In structural beams, the effect of a knot on bending strength depends on the size and location of the knot. In a simply supported beam, for example, knots on the lower side are placed in tension, those on the upper side in compression and those at or near the neutral axis in horizontal shear. On the tension side at the point of maximum stress, a knot has a marked effect on the total load a beam will carry, while knots on the compression side are somewhat less serious. In any location, knots have little effect on shearing strength.

Knots have little or no effect on stiffness; hence, in long posts where stiffness is the controlling factor knots are not viewed as a strength reducing defect. In short and intermediate posts, the reduction in strength due to knots is approximately proportional to the size of the knot. Large knots, however, have a somewhat greater relative effect than small knots.

The reduction in strength due to the presence of knots in a given piece of lumber is caused primarily by local distortion of the wood grain in and around the knot. Knots interrupt the normal direction of the grain and cause localized cross grain with very steep slopes. As knot size increases, the distortion of the grain around the knot is more than proportionally increased, therefore, the size of a knot compared to the size of the piece is an important consideration.

Since the strength reducing effect of a knot depends more on the distortion of the surrounding grain than on the knot itself, knot holes have the same effect on strength as knots. Since holes due to other causes are not accompanied by a distortion of the grain, the limitations that apply to knot holes are sufficient to determine the effect of other holes as well.

A-4.03 Checks, Shakes, and Splits

Checks, shakes, and splits are defined as follows:

- Check is a separation of the wood fibers along the grain but across the rings of annual growth. Checks do not pass through the entire thickness of the wood.
- Shake is a separation of the wood fibers along the grain between and parallel to the rings of annual growth. Shakes often extend along the face of boards and sometimes below its surface.
- Split is a lengthwise separation of the fibers extending from one surface completely through a piece to another surface. Splits commonly occur at the ends of wood members.

Checks commonly occur as a result of unequal shrinkage during seasoning. Shakes are the result of the rupture of wood cells in a weakened portion of the wood; they seldom develop unless they were present to some degree before the tree was felled.

Checks and shakes will reduce the shearing strength of members subject to bending, particularly if they are located near the neutral axis. They do not have a significant effect on the strength of members subject to compression parallel to or perpendicular to the grain.

Splits are the result of internal stresses or rough handling. Splits affect strength in the same way as checks and shakes.

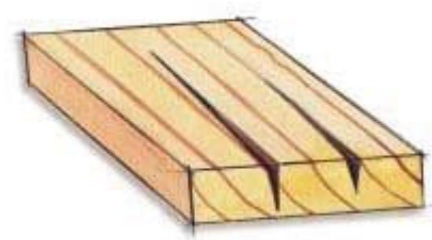


Figure A-1. Check

Figure A-2. Shake

Figure A-3. Split

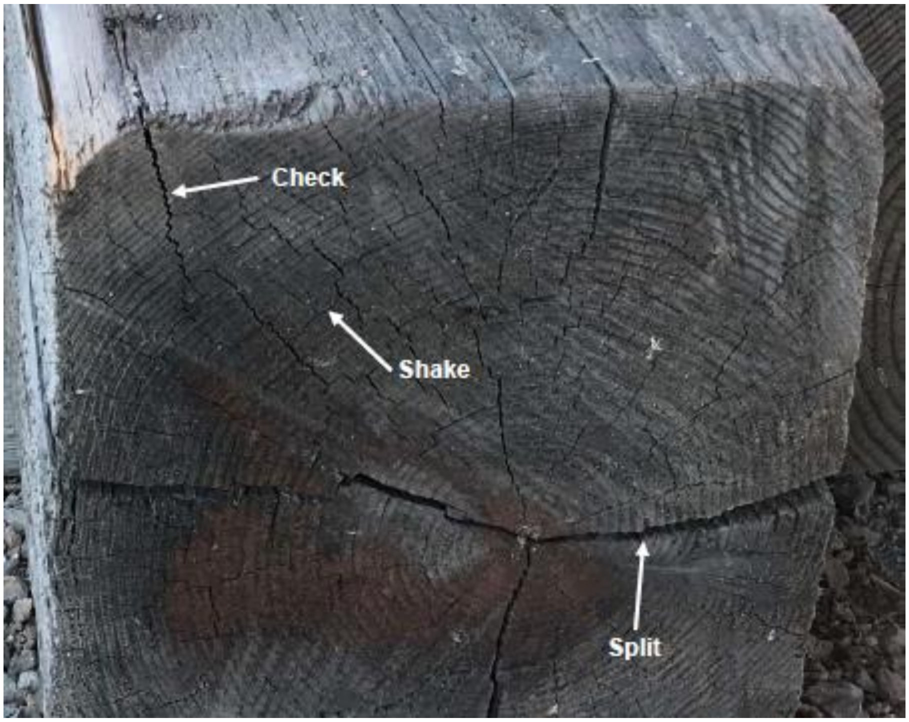


Figure A-4. Checks, Shakes, and Splits

A-4.04 Cross Grain

Cross grained wood is defined as wood in which the cells or fibers run at an angle with the axis, or sides, of the piece.

To determine the effect of cross grain on the strength of wood, it is necessary to have some measure of its degree. This is afforded by the slope of the cross grain, which is defined as the deviation of the grain from the edge of the piece or from a line parallel to its principal axis. Slope is usually designated by the ratio of a 1-inch deviation of the grain from the edge to the distance along the edge over which the deviation occurs. Thus, a slope of 1-in-20 means that over a distance of 20-inches along the edge, the grain deviates 1-inch from the edge.

Since the ratio of tensile strength parallel to the grain to tensile strength at right angles to the grain ranges from about 25 to 1 in unseasoned wood to as high as 45 to 1 in air dry material, it is apparent that even the slightest deviation from straight grain will tend to reduce the tensile strength of a given piece of lumber. However, this decrease does not become appreciable until a slope of about 1-in-20 is reached. Hence lumber in which the slope of the grain is less than 1-in-20 is considered as straight grained material for all practical purposes.

In compression, the effect of cross grain is less marked since strength parallel to the grain in both unseasoned and dry material is only about three to five times the strength perpendicular to the grain. Therefore, the slope must approach 1-in-10 before a decided decrease in compressive strength is evident.

In shear, the weakening effect of cross grain is small and usually is neglected.

A-4.05 Diagonal Grain

Diagonal grain is produced in lumber entirely by the method of sawing and has no reference to the natural alignment of the wood elements. In cutting lumber, if the plane of the saw blade is not approximately parallel to the bark surface, the grain of the wood will not be parallel to the edges and thus is termed diagonal.

Diagonal grain has the same strength reducing effect on a piece of lumber as cross grain.

A-4.06 Warping

Warping is defined as any deviation of a piece of lumber from a true or sawn surface. Warping most often occurs as a result of differences in the longitudinal shrinkage of the two faces of a board. It also may be caused by internal stresses present in the log at the time of sawing.

Warping has no effect on the inherent strength of wood, however, pronounced warping will materially reduce the bearing area of joists and beams and thus make it difficult to develop a satisfactory connection.

A-4.07 Wane

Cut lumber is sometimes characterized by the presence of bark, or by a lack of wood, on the otherwise square edges or corners of a piece. This condition, which is termed wane, is commonly considered a defect although it has no direct effect on strength except as it reduces the cross-sectional area of the piece.

As with warping, wane present at the end of a piece will reduce the bearing area and thus indirectly increase the bearing stress.

A-4.08 Decay

Wood is subject to attack by many low forms of plant life known as fungi. These wood inhabiting fungi differ from ordinary green plants in form, lack of green coloring matter, and methods of nutrition. Unlike green plants, they are unable to manufacture their own food, but must have organic material already prepared for their use. This they find in the wood substance composing the cell walls. The action of the fungi results in disintegration of the actual wood substance and gives rise to the condition known as decay.

The development of decay is dependent on the presence of an appreciable amount of moisture in the wood. Although the minimum requirements vary with different fungi, it is generally considered that wood must contain at least 20% moisture before decay will occur. Therefore, thoroughly air dried or kiln dried lumber is immune from decay unless it is subjected to wetting over a long enough period of time that its moisture content is raised to approximately the fiber saturation point.

A small supply of oxygen is necessary for the fungi to grow and develop, so that wood that is completely saturated is immune to decay.

Since decay involves an actual breaking down of the cell walls, it is evident that it vitally affects the strength of wood, particularly in the advanced stage. Never use decayed lumber for any structural purpose.

A-4.09 Marine Borer

Timber piles, when exposed for an extended period to a salt water environment, have been damaged by marine borers.

Appendix B: Falsework Reminder List

Table of Contents

B-1 Introduction

This appendix lists the location of falsework information required for the review, authorization and inspection of falsework. The information referenced addresses safety, submittal of shop drawings, design criteria, erection and removal of falsework. This reminder list is not intended to be an all-inclusive comprehensive list but is a guideline in which the structure representative can base a competent review.

B-2 Location of Falsework Information

B-2.01 Falsework Manual:

- Web link: https://dot.ca.gov/programs/engineering-services/manuals
- Chapter 1, *Introduction*: Purpose, design methodology, contractual relations, etc.
- Chapter 2, *Review of Shop Drawings*: Review procedure for shop drawings including when railroad is involved.
- Chapter 3, *Loads*: Loads on falsework.
- Chapter 4, *Design Considerations*: Various considerations for falsework, including camber, prestressing, various bridge types, openings at roadways and railroads, etc.
- Chapter 5, *Analysis*: Analysis of falsework members.
- Chapter 6, *Stability*: Falsework overturning, collapse, and bracing.
- Chapter 7, *Manufactured Assemblies*: Testing and application of commercial and non-commercial assemblies including vertical shoring systems.
- Chapter 8, *Foundations*: Analysis of foundation members, pads, piles, etc.
- Chapter 9 *Inspection*: Inspection of falsework members.

B-2.02 Special Provisions:

- Project specific requirements and considerations
- 12, *Temporary Traffic Control*: Project specific specifications for traffic openings

B-2.03 Standard Specifications:

- 5-1.20C, *Railroad Relations*: Railroad relations are available in the Information Handout.
- 5-1.23, *Submittals*: General specifications for submittals and shop drawings.
- 7-1.04, *Public Safety*: Specifications for public safety, including:

- Paved passageway or wooden walkway.
- Vertical clearance sign for vertical clearance of 15.5-feet or less.
- Do not move or temporarily suspend anything over a traffic lane open to the public unless the public is protected.

- 12-3, *Temporary Traffic Control Devices*: Specifications for traffic handling equipment and devices including K- rail.
- 48-1, *General*: General specifications for all temporary structures.
- 48-2, *Falsework*: Specifications for falsework.
- 48-2.01C, *Submittals*: Specifications for falsework submittals.
- 48-2.01C(2), *Shop Drawings*: Specifications for falsework shop drawings.
- 48-2.01D(2), *Welding and Nondestructive Testing*: Specifications for welding falsework.
- 48-2.03B, *Foundations*: Specifications for falsework foundations and pile driving.
- 48-2.03E, *Falsework Lighting*: Specifications for falsework lighting.
- 51-1.03C(2), *Forms*: Specification for deflection of forms.
- 51-1.03D(2), *Concrete Bridge Decks*: Requirement for placing concrete decks.
- 55-1.03B, *Falsework*: Specifications for falsework supporting steel structures.

B-2.04 Bridge Construction Records and Procedures Manual:

- Volume 1:
  - BCM B-2.05, *Emergency Operations Plan (EOP)*: Opening or closing of road or structure.
  - BCM C-4.14, *Notice of Change in Structure Clearance or Permit Rating*: Reporting changes is clearance. Submit notification, form TR-0019 and TR-0020 or TR-0029.
- Volume 2:
  - BCM 48-2, *Temporary Structures – Falsework*: Submitting falsework shop drawings, including railroad.
  - BCM 51, *Concrete Structures*: Lost deck forms and soffit forms.
  - BCM 75, *Miscellaneous Metal*: For permanent work.
  - BCM 50-1.03B, *Prestressing Concrete – General – Constructions – Prestressing*:

- BCM 52, *Reinforcement*: Used in concrete pads.
- BCM 180, *Welding*: For permanent work.

B-2.05 Cal-OSHA – Title 8 Regulations:

- Chapter 3.2, *California Occupational Safety and Health Regulations*, Subchapter 2 *Regulations of the Division of Occupational Safety and Health*, §341 *Permit Requirements*: Permits for falsework.
- Chapter 4, *Division of Industrial Safety*, Subchapter 4 *Construction Safety Order*, §1717 *Falsework and Vertical Shoring*. Requirements for submittals and construction of falsework and vertical shoring.

B-2.06 SC HQ Falsework Engineer:

- (916) 227-8060

B-3 Pre-Job Meeting

It is recommended that the structure representative prepares to address these items for discussion in the pre-job meeting.

B-3.01 Falsework Design Review and Authorization:

- Review time allowed.
- Review time starts when a complete submittal is received.
- Review time starts over when corrections or revisions are made.
- Priority listing for multiple submittals.
- Information required for complete submittal. Refer to Section 2-4.01, *Initial Review.*
- For proprietary products and manufactured assemblies, the engineer should request technical data to be included in the submittal. Items include hardware items such as overhang brackets, jacks, hangers, concrete inserts, finishing machines, etc., and all commercial shoring systems.
- For cable bracing systems, the engineer should request manufacturer's technical data to be included in the submittal.
- Falsework erection cannot begin until drawings are authorized except for pad and pile foundation work.
- Request meeting with the falsework designer, foreman, SC engineers and other key individuals prior to falsework erection, grading and removal operations.

- Request that a joint safety stand down be held after falsework incident in the event it should happen.
- Verify Cal-OSHA Title 8 Regulations considerations are addressed
- Verify Storm Water Pollution Prevention Plan requirement are addressed

B-3.02 Falsework Erection and Removal Plans:

- Project specific considerations
- Erection and removal procedure must be shown on the shop drawings.
- See Section B-9, *Erection Plan Check List* and Section B-10, *Removal Plan Check List.*

B-3.03 Welding and Welded Connections:

- All welds must comply with AWS. Certified welder is required.
- Splice welds require NDT
- For previously welded splices all necessary testing and inspection must be documented to certify the ability of the weld to sustain the design stresses.

B-3.04 Traffic Considerations:

- Traffic openings, vehicle, pedestrian, railroad, etc.
- K-rail
- Lighting
- Detours, closures
- Erection and removal. Cannot erect or remove falsework over traffic.

B-3.05 Railroad Involvement:

- Railroad requirements
- Shop drawing submittal procedure and review time where railroad is involved

B-3.06 Application of Construction Safety Orders:

- Cal-OSHA Title 8 Regulations

B-3.07 Deck Placement Plan:

- *Bridge Deck Construction Manual*

B-4 Design Loads

B-4.01 Vertical Design Loads:

- Minimum vertical design load (dead load plus live load) on any falsework member is 100 psf. This includes supports for a construction walkway extending beyond edge of deck or for bent cap falsework.
- For stress analysis, the design dead load is the weight of concrete, forms, reinforcing steel and falsework members.
- For deflection, the design dead load is the weight of concrete only, i.e. without forms and rebar dead weight.
- Design live load includes the following:
  - 20-psf over the total area supported by the member under consideration.
  - 75-plf at edge of deck overhang acting over a maximum length 20-feet. Does not need to be applied simultaneously with Bidwell on deck overhang brackets.
  - Weight of equipment (finishing machine, etc.) applied as a concentrated load at point of contact.

B-4.02 Horizontal Design Load:

- The assumed horizontal load is the greater of the following:
  - Sum of the actual loads due to equipment, construction sequence or other causes and wind loading.
  - 2% of the total supported dead load of the bridge during the unloaded and loaded condition.

B-4.03 Miscellaneous Loads:

- Increased vertical design load adjacent to roadway and railroad.
- Increased vertical load due to load redistribution caused by prestressing forces.
- Horizontal load caused by stream flow pressure. See Caltrans *Trenching & Shoring Manual* for hydrodynamic forces.
- Loads due to vertical and horizontal components of cable loads.

B-5 Shop Drawings

The items in this list should be shown on the shop drawings. This list is not all inclusive but is intended as a guide when reviewing the shop drawings.

All items listed in Section 2-4.01, *Initial Review.*

Anticipated settlement, not to exceed 1-inch.

B-5.01 Pads:

- Assumed soil bearing value for pad foundations
- Joint location in continuous timber pads
- Design details for concrete pads

B-5.02 Piles:

- Diameter
- Section type for steel piles
- Design details for concrete piles
- Pile tip and resistance
- Driving tolerances, maximum pile-pull, and eccentricity

B-5.03 Dimensions:

- Bent locations
- Post heights
- Post spacing
- Span lengths
- Stringer spacing
- Vertical distance between connections in diagonal bracing
- Size of all load supporting members

B-5.04 Timber Bracing:

- Type of connection (single or double shear)
- Type, size, and number of fasteners at each connection

B-5.05 Cable Bracing:

- Cable description, number and size of cables in each cable brace
- Number and type of connectors (Crosby clips, etc.)

- Detail showing method or device by which cable will be attached to falsework components, and location of attachment
- Cable anchorage
- Cable preload value and method by which preload force will be applied and measured
- Constructional stretch considerations
- Adjustment plan for grading and adjusting falsework more than 1/2-inch
- Cable adjustment and release procedure during grading and adjustment

B-5.06 Welds:

- All welds must comply with AWS

B-5.07 Commercial Shoring Systems:

- For commercial shoring systems, the trade name and nominal load-carrying capacity

B-5.08 Erection and Removal Plans:

- The method or procedure to be followed, including details for temporary bracing

B-6 Design Considerations

B-6.01 General

B-6.01A Loads:

- Loaded zone
- Differential beam deflection considered
- Additional requirements for deck overhangs on steel girder bridges
- Steel girder widening falsework independent of existing bridge
- Load redistribution due to application of prestressing forces

B-6.01B Foundations:

- Soil bearing value compatible with site conditions. Soil load test required.
- Pad joint location
- Corbel spacing

- Bearing on timber piles limited to 45 tons
- Pile bents:
    - Driving tolerance
    - Required penetration
    - Bracing
    - Horizontal deflection
    - P-delta deflection
    - Longitudinal stability

B-6.01C Posts and Columns:

- Timber post L/d
- Steel post L/r
- Steel crush plate between timber post and timber cap

B-6.01D Commercial Shoring Systems:

- Manufacturer's technical data furnished and reviewed
- Design loads comply with manufacturer's recommendations for all loading conditions
- Shoring designed in accordance with manufacturer's recommendations and falsework manual design criteria
- Cable bracing connected to cap at top and to external support at bottom
- Cable design load meets falsework manual criteria

B-6.01E Bracing:

- Diagonal bracing members and connections
- Timber members sized to accommodate number of fasteners
- Fastener capacity values adjusted for load duration
- Timber bracing compression members and compression connections adjusted to 1/2 the design value
- Connection at center of cross bracing
- Steel bracing, welded connections meet design criteria
- Cable bracing, review manufacturer's technical data. Perform load test if required

- Cable bracing, cable attached to falsework cap, not posts or columns. Check cable anchorages for uplift.
- Cable can only be used for single tier bents

B-6.01F Beams and Stringers:

- Joist stresses at girder flares, diaphragms, and caps
- Timber beams stable against buckling and rollover
- Steel beam compression flange buckling
- Steel beam bi-axial bending
- Steel beam web yielding
- Camber strips centered on stringers and within compression stress limit
- Beam deflection limited to L/240 under weight of concrete only
- Continuous beams, effect of beam continuity and beam uplift

B-6.01G Stability:

- Collapse
- Overturning
- Friction to resist horizontal forces
- Grading or adjustment:
  - Adjustment plan
  - Adequate space for jacks
  - Bearing

B-6.01H Plywood:

- Deflection limits

B-6.01I Erection and Removal Plans:

- See Section B-9, *Erection Plan Check List,* and Section B-10, *Removal Plan Check List.*
- Falsework components stable during all stages of erection and removal
- Temporary bracing (including connections) meets minimum design load criteria
- Access for removal equipment after new bridge is built

B-6.01J Miscellaneous Considerations:

- Proprietary products used in accordance with manufacturer's recommendations, manufacturer's technical data furnished and reviewed
- Manufacturer's technical data is required for all proprietary products used in the falsework, and for all cable installations
- Ledger connection for lost deck forms

B-6.02 Adjacent to Roadways

B-6.02A Clearances:

- Check Horizontal and Vertical Clearances:
  - Roadway: Shop Drawings: H __________ V __________
    Special Provisions: H __________ V __________
    Bridge Plans: H __________ V __________
  - Pedestrian: Shop Drawings: H __________ V __________
    Special Provisions: H __________ V __________
    Bridge Plans: H __________ V __________
- Openings conform to table in Special Provisions 12-4, *Maintaining Traffic*
  - Vertical clearance sign required for vertical clearance of 15.5-feet or less, see Standard Specifications 7-1.04, *Public Safety*
  - K-rail length and clearance to falsework adequate

B-6.02B Posts:

- Steel or timber with minimum section modulus about each axis:
- Post design load is greater of:
  - 150% of normal post loading
  - Increased or readjusted loads caused by prestressing

B-6.02C Bracing:

- 5/8-inch diameter or larger bolts for timber bracing connections

B-6.02D Mechanical Connections to Resist Impact:

- 2000 lb. capacity at base of posts in all directions except toward the roadway
- 1000 lb. capacity top of post all directions

- 500 lb. capacity for certain stringer-to-cap connections effective in all directions including uplift

B-6.02E Falsework Lighting:

- Lighting Plan
- Portal lighting and white panels
- Roadway illumination
- Pedestrian walkway lighting

B-6.02F Pedestrian Openings:

- Paved passageway or wooden walkway see *Standard Specifications*, Section 7-1.04, *Public Safety*
- Handrail per Cal-OSHA requirements
- Overhead debris protection

B-7 Adjacent to Railroad

B-7.01 General:

- Requirements listed in the Information Handout *Railroad Relations*

B-7.02 Shop Drawings:

- Construction features affecting railroads require approval by the railroad company

B-7.03 Clearances:

- Check Horizontal (H) and Vertical (V) Clearances:
  - Railroad: Shop Drawings: H __________ V __________

    Special Provisions: H __________ V __________

    Bridge Plans: H __________ V __________
- Vertical clearance measured from top of rail. Consider beam deflection and settlement
- Horizontal clearance measured from centerline of tracks
- Complete railroad checklists. See Temporary Structure Technical Team website

B-7.04 Posts:

- Steel or timber with minimum section modulus about each axis.
- Post design load is the greater of:
  - 150% of normal post loading
  - Increased or readjusted loads caused by prestressing

B-7.05 Bracing:

- 5/8-inch diameter or larger bolts for timber bracing connections

B-7.06 Mechanical Connections to Resist Impact:

- 2000 lb. capacity at base of posts in all directions except toward the roadway
- 1000 lb. capacity top of post all directions
- 500 lb. capacity for all stringer-to-cap connections effective in all directions including uplift

B-7.07 Bents Within 20 Feet of Track Centerline:

- Solid sheathing 5/8-inch plywood or 3/4-inch thickness lumber between 3 and 17 feet above track on track side of bent
- Bracing designed to resist the horizontal design load, but not less than 5000 lb.

B-8 Authorizing Shop Drawings

B-8.01 General:

- Review the shop drawings and perform an engineering analysis
- Stamp each shop drawing sheet with the Caltrans Authorization stamp. Structure representative or licensed engineer who reviewed the shop drawings signs and dates the stamp
- Structure representative or licensed engineer who reviewed the shop drawings stamps, signs, and dates temporary structure analysis report
- See also BCM 48-2, *Temporary Structures – Falsework*

B-8.02 When Railroad Company is Involved:

- Review the shop drawings and perform an engineering analysis
- Complete railroad checklists. See Temporary Structure Technical Team website

- After reviewing the shop drawings, but before authorizing them, send the shop drawings and the check list to the SC HQ Falsework Engineer who will forward them to the railroad for approval
- Do not authorize the shop drawings until notified by SC HQ Falsework Engineer that the railroad has approved the shop drawings
- See also BCM 48-2, *Temporary Structures – Falsework*

B-9 Construction Considerations

B-9.01 Erection Check List:

- See Section B-10, *Erection Plan Check List*

B-9.02 Erection Plan:

- Before erection begins, review the erection plan with State and contractor personnel
- For stage construction, effect of erection and location of other stages considered

B-9.03 Pad Foundations:

- Foundation material adequate to support design soil pressure
- Soil bearing test needed
- Splices in continuous pads located properly
- Pads protected from flooding and surface runoff

B-9.04 Pile Foundations:

- Required pile resistance obtained
- For pile bents: penetration and driving tolerances meets design assumptions

B-9.05 Timber Construction:

- Timber quality
- Connections conform to design details
- Connectors properly installed
- Workmanship

B-9.06 Manufactured Assemblies:

- All commercial products and devices used and installed in accordance with manufacturer's recommendations
- Certifications furnished

B-9.07 Metal Shoring Systems:

- Assembly meets manufacturer’s recommendations
- Certifications furnished

B-9.08 Cable Bracing:

- Cable is same size and type as shown on shop drawings
- Connections conform to shop drawing details
- Crosby clips properly installed and torqued
- Cable preload force applied
- Cable preload force applied twice for cables attached to timber members

B-9.09 Falsework Openings:

- Clearance notification:
  - Contractor to notify resident engineer no less than 25 days and no more than 125 days before operation
  - Structure representative notifies the resident engineer who notifies the Transportation Permits Board. See Section 4-12.02, *Falsework Openings*
  - Re-notify after erection if actual clearance is different
  - If clearance is less than notified, stop operations and remove the stringers already set until clearance issues are resolved
- White panel boards properly positioned
- Lighting inspected after dark

B-9.10 Field Changes:

- All changes must be documented and resubmitted for authorization, see *Falsework* Manual, Section 9-3.22, *Field Changes*

B-9.11 Certification:

- See *Falsework* Manual, Section 9-3.26, *Falsework Certification*

- Certification of falsework by licensed engineer
- Certification of manufactured assemblies including shoring systems

B-9.12 Inspection During Concrete Placement:

- See *Falsework* Manual, Section 9-4.01, *Inspection During Concrete Placement*
- Contractor must follow the deck placement plan
- Inspect falsework at frequent intervals during concrete placement. Look for the following indicators of incipient failure:
    - Excessive compression at the tops and bottoms of posts and under the ends of stringers. Crushing of wedges. Settlement of sand jacks
    - Movement or deflection of diagonal bracing. Distortion at connections Pulling of nails
    - Tilting or rotation of joists or stringers. Excessive deflection of any horizontal member
    - Posts or towers that are bowing or moving out of plumb
    - Excessive settlement as indicated by telltales
    - The sound of falling concrete or breaking timbers. Any unusual sound
- Inspect tell tales
- Settlement must not deviate more than ±3/8-inch from the anticipated settlement on the shop drawings

B-9.13 Inspection After Concrete Placement:

- See *Falsework* Manual, Section 9-4.02, *Inspection After Concrete Placement*
- Foundation protected from undermining by curing water
- Foundation protected from undermining by rain water

B-9.14 Deck Shrinkage:

- Deck shrinkage during curing can redistribute loads toward the center of the bridge spans
- Post tensioning of bridges will redistribute loads toward the bridge supports. This can also affect the falsework loads.

B-9.15 Falsework Removal:

- See *Falsework* Manual, Section 9-5, *Removal*

- See Section B-10, *Removal Plan Check List*
- Review removal plan with State and contractor personnel
- Falsework components stable during all stages of removal
- Effect of temporary unbalanced and/or eccentric loads
- Effect of jacking loads
- Effect of crane set on the permanent structure
- Winch loads on new bridge
- For stage construction, effect of removal sequence considered

B-10 Erection Plan Check List

B-10.01 Items That Should be Included in the Erection Plan

B-10.01A Information (for example) to be Provided as Part of the Falsework Erection Sequence:

- Falsework pad grading:
  - Verify soil capacity
  - Provide for drainage
- Method of falsework bent construction:
  - Type of equipment used for erection
  - Location of equipment used for erection
  - Material storage areas
- Sequence of falsework bent erection. For example, the plan could state:
  - First erect and stabilize bents at columns and abutments
  - Secure top and bottom of bents at column or abutment
- Install stability measures (bracing) as indicated on the approved falsework plans before placing stringers:
  - Temporary and permanent stability measures are to be shown on the authorized falsework plans
- Order of stringer erection. Are interior or exterior stringers placed first? Be aware of stringers on the cantilever portions of bents.
- Sleeper and camber strip placed on the stringer prior to or after stringer erection

B-10.01B Notes (for example) Stating:

- Falsework bents are to be stable at all stages of erection. Details of interim stability measures are shown on the plans
- All permanent stability measures shall be in place before erecting falsework members above the stringers
- Secure stringers prior to soffit joist or panel placement
- Where bolts are required for permanent bracing, nails may be used as a temporary measure

B-10.01C Safety Measures:

- Details provided for workers rolling out soffit joists
- Details provided for soffit form or panel placement. Including measures for possible high winds
- Details provided for exterior girder panel placement. Including measures for possible high winds

B-10.01D Adjustment Including Grading:

- Details of falsework grading procedure
- Adjustment plan if adjustment is over 1/2-inch

B-10.01E Special Locations:

- See Standard Specifications 48-2.02B(4), *Special Locations*

B-10.01F Falsework over Roadways:

- Verify that the plan adequately addresses time available for erection. Refer to lane closure charts in the contract Special Provisions.
- Verify that permanent bracing is installed, and falsework is stable prior to allowing traffic to pass through falsework.

B-10.01G Falsework over Railroad:

- See Railroad Relations in the Information Handout
- See Union Pacific Railroad (UPRR) – BNSF Railway, *Guidelines for Railroad Grade Separation Projects.* Available on the UPRR or BNSF websites. The most current version always applies regardless of which one is listed in the Information Handout.

- Verify that the plan adequately addresses time available for erection. Refer to railroad relations in the Information Handout
- Verify that permanent bracing is installed, and falsework is stable prior to allowing traffic to pass through falsework

Review *Standard Specifications*, Section 48-2.03C, *Erection,* for other requirements.

B-11 Removal Plan Check List

B-11.01 Items That Should be Included in the Removal Plan

B-11.01A Sequence of Falsework Removal:

- Order in which falsework spans, bents, stringers, and formwork will be lowered and removed

B-11.01B Considerations for Load Redistribution Due to Pre-stressing:

- How is the load redistribution determined?
- Effect on individual falsework bents
- Effects on the removal sequence

B-11.01C Method of Falsework Release:

- How is falsework released from the structure?
- Sand jacks being used
- Effect of falsework release on bracing
- Details and description on how falsework stability is maintained

B-11.01D Notes Stating:

- Falsework will be stable during all phases of removal. Specific stability measures at all stages of removal should be outlined

B-11.01E Indicate (for example) on Plan:

- Equipment to be used
- Location of equipment during various stages of removal
- Sequence of stringer removal
- Lay down areas for materials removed
- How falsework stability is maintained throughout falsework removal process

- Person in charge to be on site during falsework removal operations
- Number and function of people required onsite to safely remove falsework

B-11.01F Winch Systems:

- *Standard Specifications*, Section 7.1.04, *Public Safety,* states: "Do not move or temporarily suspend anything over a traffic lane open to the public unless the public is protected."
- Submittal should include details regarding:
    - Winch placement
    - Winch capacity, loads, and dead men required
    - Winch cable connection to falsework
    - Supplemental and redundant support system
    - Patching details for holes through deck and soffit
- Falsework may not be supported by winches over traffic. Another independent support system is required

B-11.01G Special Locations:

- See Standard Specifications 48-2.02B(4), *Special Locations*

B-11.01H Falsework over Traffic:

- Verify that the plan adequately addresses time available for removal. Refer to lane closure charts in the contract Special Provisions.
- Verify that temporary bracing is installed, and falsework is stable prior to releasing permanent bracing.
- If falsework is to be released and lowers the vertical clearance, estimate and report changes to impaired vertical clearances prior to the operation.
- Provide contingency plans for falsework mishap.

B-11.01I Falsework over Railroad:

- See Railroad Relations in the Information Handout
- See Union Pacific Railroad (UPRR) – BNSF Railway *Guidelines for Railroad Grade Separation Projects.* Available on the UPRR or BNSF websites. The most current version always applies regardless of which one is listed in the Information Handout.
- Verify that the plan adequately addresses time available for removal. Refer to railroad relations in the Information Handout.

Appendix C: Falsework Memos

C-1 Introduction

This appendix lists Falsework Memos providing additional guidance and clarification associated with design and construction of falsework.

Appendix D: Example Problems

Table of Contents

Appendix D Example 1 – Clearances at Falsework Openings

Prior to falsework erection the theoretical impaired clearance is calculated to provide advance notice to the Resident Engineer (RE) and Traffic Operations. This calculation will require determining clearance between the bridge and the roadway and the clearance under the falsework. When the stringers are placed, vertical clearance shall be physically measured to ensure that vertical clearance is equal to or greater than the reported vertical clearance. The measured vertical clearance needs to be reported to Resident Engineer and Traffic Operations. Note both vertical and horizontal clearances are required to be reported in TR-0029 form.

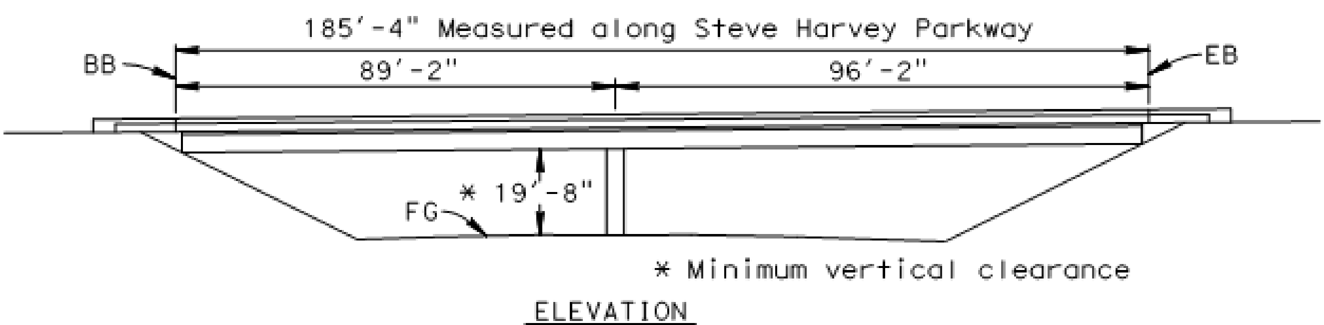
Determine the impaired clearance for the bridge and the falsework configuration given below.

Given Information

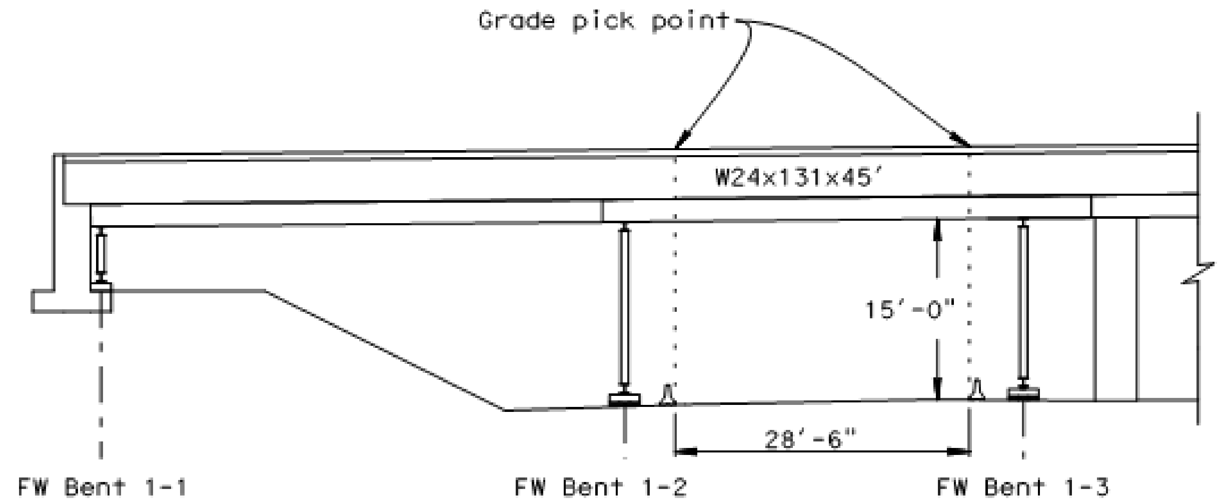
General Plan:

Pick the Point of Minimum Clearance = 19'-8" (236.00").

(Note: This point is the minimum clearance between the bridge and the finished roadway without the falsework. It should be noted that during construction due to overlay, stage construction, or roadway profile changes, the point of minimum clearance and location may be different from that shown on the project plans.)



Falsework Plans

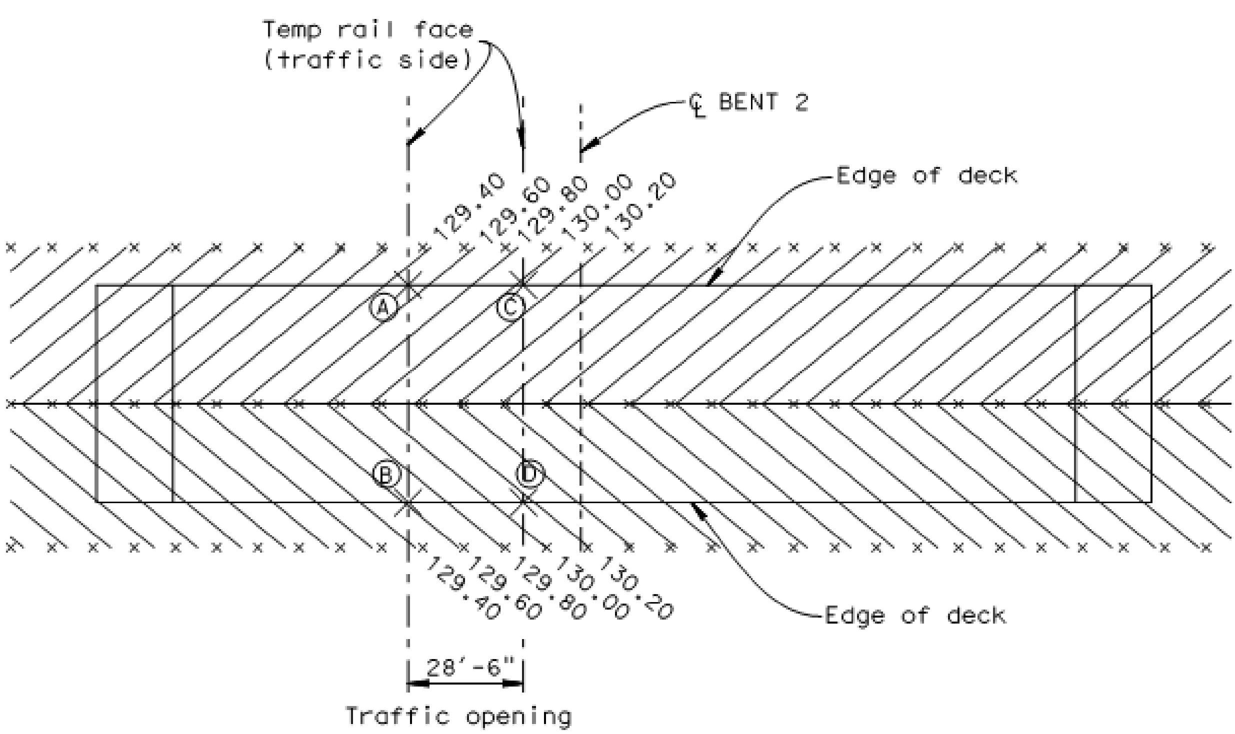


PARTIAL FALSEWORK ELEVATION

Special Provisions

Vehicle openings: 20'-0" wide and 15'-0" height.

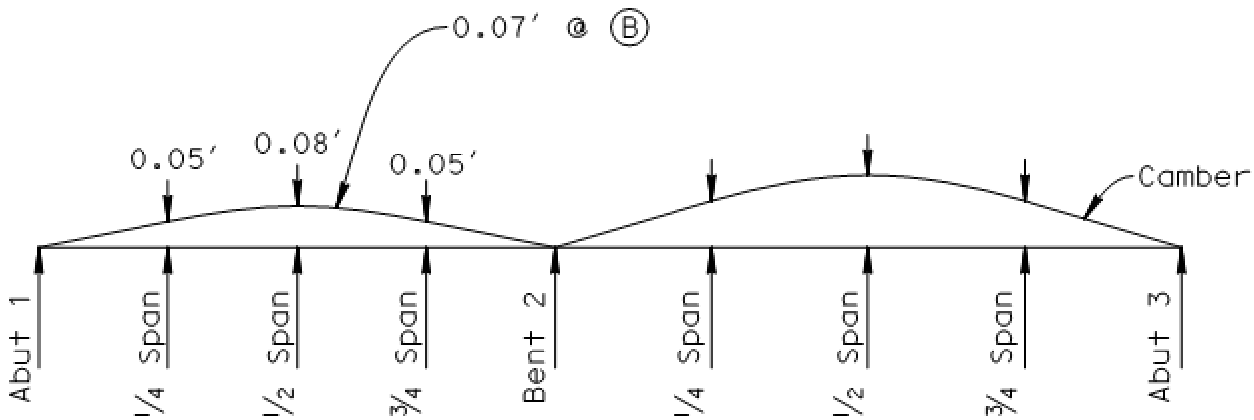
Deck Contour Sheet (4-Scale):



PLAN

Points A, B, C and D grades are taken at the edge of deck at the temporary railing face.

Bridge Camber Diagram:



CAMBER DIAGRAM

Check Vertical Clearance

1. Calculate vertical clearance between bridge and roadway:

   Points A, B, C and D are the edge of deck grade above the four corners of the traffic opening defined by the face of temporary rail and edge of deck.

   Determine the elevation of the pavement (roadway grade) by field surveying below the points described above. The number of plotted points can be more than four for complex layout. Note in the following table, bridge camber value determined by plotting on the 4-scale at the falsework bent is included.

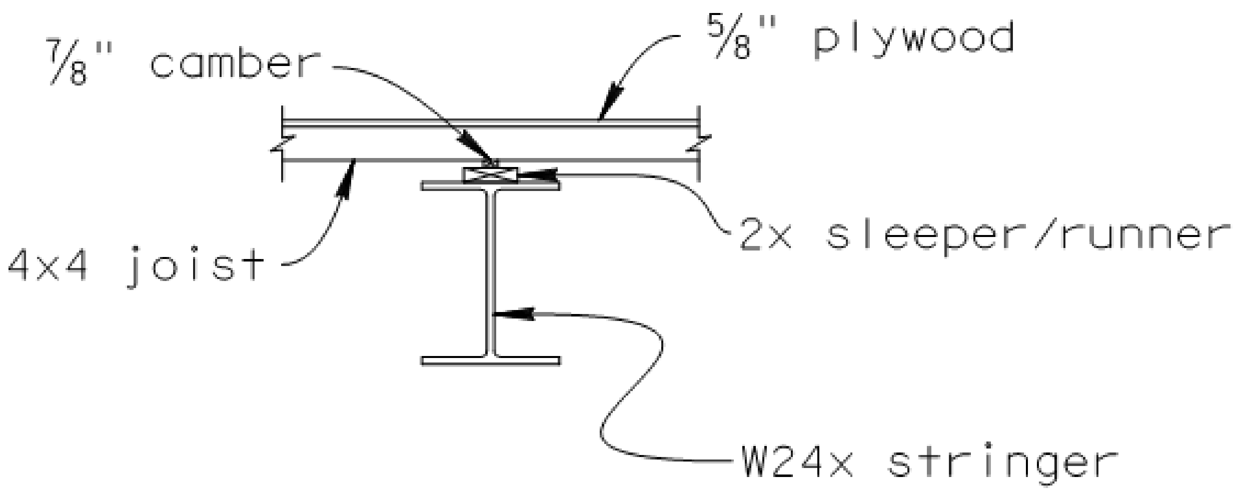
Point	Deck Grade (ft)	Roadway Grade (ft)	Box Girder Depth (ft)	Bridge Camber (ft)*	Clearance (ft)	Clearance (inch)
A	129.50	106.00	4	+0.07	19.57	234.84
B	129.62	106.70	4	+0.07	**18.99**	**227.88**
C	130.04	104.34	4	+0.01	21.71	260.52
D	130.12	105.96	4	+0.01	20.17	242.04

The minimum vertical clearance between the bridge and the roadway at the traffic opening is Point B with 227.88" (18'-11 7/8") height.

* Camber is additive for positive camber.

2. Calculate vertical clearance between falsework and roadway:
   a. Calculate Minimum Vertical Clearance:

Bridge Clearance from table above	= (+) 227.88"
Falsework Depth	
Plywood = 5/8"	= (-) 0.63"
Joist (4 x 4)	= (-) 3.50"
Camber (0.07')	= (-) 0.84"
Runner (2x wood)	= (-) 1.50"
Stringer (W24X131)	= (-) 24.48"
Calculated Minimum Vertical Clearance = 196.89" (16'-4 7/8")	



FALSEWORK PLANS

   b. Subtract Pavement elevation changes (- or 0):

   If the roadway under the bridge is to be paved prior to removal of the falsework, the net thickness of overlay will need to be subtracted from the clearance. The net thickness accounts for any grinding that may take place prior to paving.

   $\Delta_b = 0$ (For no paving)

   c. Subtract Adjustment of Falsework grades (- or 0):

   Often contractors set the falsework bent lower prior to final grading. In that case subtract the value. If the falsework is kept higher than theoretical value and then adjusted downward a value of zero must be used.

$\Delta_c = 0$ (In this example no FW adjustment)

d. Subtract Falsework settlement (-):

The probable or anticipated settlement of the falsework per falsework plans. Note theoretically, this value can be zero because the falsework will be erected higher to account for settlement.

$\Delta_d = -0.8"$ (It is conservative to include)

e. Subtract Falsework stringer deflection (-):

Note that stringer will deflect even with the use of camber strips. Conservatively, in this calculation stringer deflection at the center of falsework span is used.

$\Delta_e = -1.13"$

f. Subtract Release of sand jacks/ wedging (-)
If traffic will be allowed under the structure after the sand jacks/wedging is blown /removed and prior to stringers being removed, then this allowance needs to be included:

$\Delta_f = -5.5"$ (Sand jacks w/ 2 X 6 side members)

g. Calculated ultimate actual clearance:

This is equal to the value of line “a” plus summation of bullet “b” thru “f”:

a) Min. Vertical clearance	**196.89"**
Subtract Bullet “b” through “f” below	
b) Pavement Surfacing	-0.00"
c) Falsework Grade	-0.00"
d) Falsework Grade	-0.80"
e) Deflection	-1.13"
f) Sand Jack	-5.50"
g) Min Vertical Clearance	= **189.46"**

Net vertical clearance height 189.46" = 15'- 9 1/2" > 15'-0"

Value is greater than or equal to that given in the Special Provisions therefore acceptable.

h, Clearance to report:

This is the value “g” rounded down to the nearest 3"

From 15'-9 1/2" to 15'-9"

Value = 15'- 9" > 15'-0"

Check Horizontal Clearance

The horizontal opening shown on the falsework plans is 28'-6" which is greater than 20'-0" given in the contract Special Provisions, Section "Maintaining Traffic," and therefore acceptable.

Summary

1. Use items a to h above to complete Form No. SC-4103, *Report of Falsework Clearance.*
2. Use 28'-6" for clear horizontal opening in Form No. SC-4103.
3. Items 1 & 2 above provide all the values required to complete Form No. SC-4103. Refer to the following BCMs to complete form Forms TR-0019 or TR-0029 when required to complete them on behalf of the Resident Engineer:
   a. BCM C-4.14, *Notice of Change in Structure Clearance or Permit Rating*
   b. BCM 120-2.0, *Impaired Clearances at Falsework Traffic Openings*
   c. BCM 120-2.1, *Reporting of Impaired Clearances at Falsework Traffic Openings*

Completed Form No. SC-4103 (Rev 12/17/13)

Department of Transportation
REPORT OF FALSEWORK CLEARANCE
Form No. SC-4103 (Formerly SC-12.6.1) (Rev. 12/17/13)

Job Stamp:
Falsework Manual
Appendix D
Example 1

Date: 1/1/2020

Bridge name: Any Bridge

Br. No. XX-XXXX

Co/Rte/PM: XXXX

Direction of travel: Northbound

Determination of falsework clearance:

a)	Calculated or Measured Minimum vertical clearance:	196.89 "
	Allowances:	
b)	Pavement elevation changes (- or 0)	0.00"
c)	Adjustment of Falsework grades (- or 0)	0.00"
d)	Falsework settlement (-)	0.80"
e)	Falsework stringer deflection (-)	1.13"
f)	Release of sand jacks (wedging) (-)	5.50"
g)	Calculated ultimate actual clearance[1]	189.46"
h)	Clearance to report[2]	15'-9"

[1] This value must be greater than that given in the Special Provisions
[2] Calculated ultimate actual clearance rounded down to the nearest 3"

The clear horizontal opening 28.5 is feet wide.

Remarks:

Appendix D Example 2 – Falsework Beam – Bi-Axial Bending – Canted ≤ 2%

This example demonstrates how to calculate maximum bending stress in beams canted less than or equal to two percent. Refer to *Falsework Manual (FW),* Section 5-4.04, *Bi-Axial Bending.*

Given Information

Span = 48 Ft   Member W 14 x 176

Cross Slope = 2% $I_{x\text{-}x}$ = 2140 in^4 $I_{y\text{-}y}$ = 838 in^4

d = 15.2 in b_f = 15.7 in.

Uniform Load W

Total Section:

Loading for stress calculations:

Load A = Dead Load (**FW 3-2.01**) + Beam Weight + LL **(FW 3-2.02- min 20 psf)**

Load A = Concrete (160 lb/ft^3) + Beam (176 lb/ft) + LL (20 lb/ft^2) = 1420 lb/ft

Loading for deflection calculation:

Load B = Concrete only (150 lb/ft^3) (**FW 3-2.01)**

Load B = Concrete only (150 lb/ft^3) = 1000 lb/ft (for calculating beam deflection)

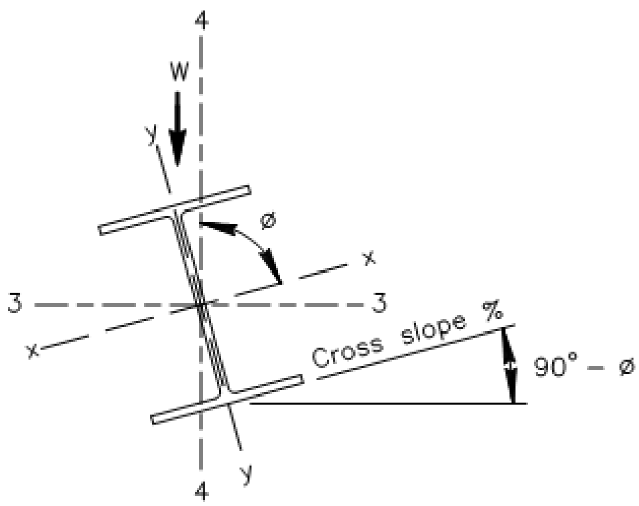
Bottom slab and stems:

Loading for horizontal calculation (when canted > 2%)

Load C = Concrete DL of Soffit Slab + Girder Stems

Load C = Concrete (150 lb/ft^3) = 649 lb/ft

Assume lateral bracing is adequate so that F_b = 22,000 psi **(FW 5-4.04)** maximum of the Standard Specifications is not exceeded.



$$Ø = 90° - \tan^{-1} \text{(cross slope)}$$

$$= 90° - \tan^{-1}\left(\frac{2.00}{100}\right) = 88.85°$$

$$y = \frac{d}{2} = \frac{15.2 \text{ inches}}{2} = 7.60 \text{ inches}$$

$$x = \frac{b_f}{2} = \frac{15.7 \text{ inches}}{2} = 7.85 \text{ inches}$$

Check Bending and Deflection

Check bending using Load A:

$$M = \frac{WL^2}{8} = \frac{1420\frac{lb}{ft}(48 \text{ Ft})^2}{8} = 408{,}960 \text{ ft-lbs} = 4{,}907{,}520 \text{ in-lbs}$$

$$f_b = M\left[\frac{y}{I_{x-x}}\sin Ø + \frac{x}{I_{y-y}}\cos Ø\right] \quad \textbf{(FW 5-4.04A-1)}$$

$$f_b = 4{,}907{,}520\left(\frac{7.60}{2140}\sin 88.85° + \frac{7.85}{838}\cos 88.85°\right) = 18{,}348 \text{ psi}$$

18,348 psi < 22,000 psi allowable **OK**

Check deflection about the 3-3 axis, using Load B:

$$\Delta_{3-3} = \frac{5WL^4}{384EI_{3-3}} = \frac{5\left(1000\ ^{Lb}/_{Ft}\right)(48 \text{ Ft})^4\left(1728\ ^{In^3}/_{Ft^3}\right)}{384\ (30 \times 10^6 \text{ psi})(I_{x-x}\sin^2 Ø + I_{y-y}\cos^2 Ø)}$$

$$= \frac{5(1000)(48)^4(1728)}{384\ (30 \times 10^6)\ (2140 \sin^2 88.85 + 838 \cos^2 88.85)}$$

$$= 1.86 \text{ In.} < \frac{L}{240} = \frac{(48)(12)}{240} = 2.40 \text{ Inches allowable} \qquad \textbf{OK}$$

Appendix D Example 3 – Falsework Beam – Bi-Axial Bending – Canted > 2%

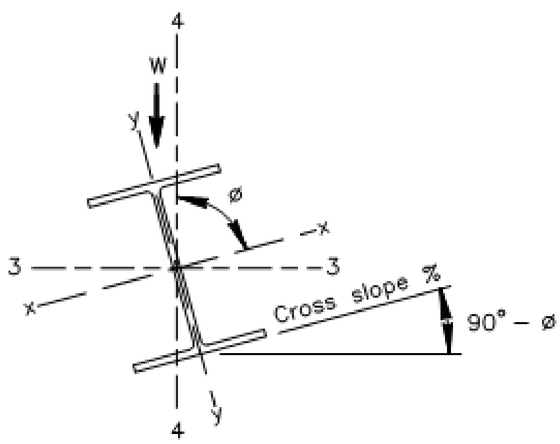
This example demonstrates how to calculate maximum bending stress in beams canted more than two percent. Refer to *Falsework Manual*, Section 5-4.04, *Bi-Axial Bending.*

Given Information

Span = 48 Ft — Member W 14 x 176

Cross slope = 10% — $I_{x\text{-}x}$ = 2140 In^4 — $I_{y\text{-}y}$ = 838 In^4

D = 15.22 In — b_f = 15.65 In



$$Ø = 90^\circ - \tan^{-1}(\text{cross slope})$$

$$= 90^\circ - \tan^{-1}\left(\frac{10}{100}\right) = 84.29^\circ$$

$$y = \frac{d}{2} = \frac{15.20 \text{ inches}}{2} = 7.60 \text{ inches}$$

$$x = \frac{b_f}{2} = \frac{15.70 \text{ inches}}{2} = 7.85 \text{ inches}$$

Uniform Load W:

Total Section:

Load A = Concrete (160 lb/ft^3) + Beam (176 lb/ft) + LL (20 lb/ft^2) = 1420 lb/ft

Load B = Concrete only (150 lb/ft^3) = 1000 lb/ft

Bottom slab and stems:

Load C = Concrete (150 lb/ft^3) = 649 lb/ft

Assume lateral bracing is adequate so that F_b = 22,000 psi maximum of the Standard Specifications is not exceeded.

$$\emptyset = 90° - \tan^{-1}\frac{10}{100} = 84.29°$$

Check Bending and Deflection

Check bending:

M = $\frac{WL^2}{8} = \frac{\left(1420\ {}^{lb}/_{ft}\right)(48\ ft)^2}{8} =$ 408,960 ft-lbs = 4,907,520 in-lbs

f_b = 4,907,520 $\left(\frac{7.60}{2140}\sin 84.29° + \frac{7.85}{838}\cos 84.29°\right)$ = 21,915 psi < 22,000 psi

allowable **OK**

Check deflections:

Check x and y deflections versus L/240 using Load B:

Load in the y-direction W_y = I000(cos(90-84.29)) = 995.04 lb/ft

$$\Delta_y = \frac{5WL^4}{384EI_{x-x}} = \frac{5\left(995.04\ {}^{lb}/_{ft}\right)(48\ ft)^4\left(1728\frac{in^3}{ft^3}\right)}{384\ (30 \times 10^6\ psi)(2140\ in^4)}$$

= 1.85 in. < $\frac{L}{240} = \frac{(48)(12)}{240} =$ 2.4 inches allowable

Load in the x-direction W_x = I000(sin(90-84.29)) = 99.49 lb/ft

$$\Delta_x = \frac{5WL^4}{384EI_{y-y}} = \frac{5\left(99.49\ {}^{lb}/_{ft}\right)(48\ ft)^4\left(1728\frac{in^3}{ft^3}\right)}{384\ (30 \times 10^6\ psi)(838\ in^4)}$$

= 0.47 in. < $\frac{L}{240} = \frac{(48)(12)}{240} =$ 2.4 inches allowable

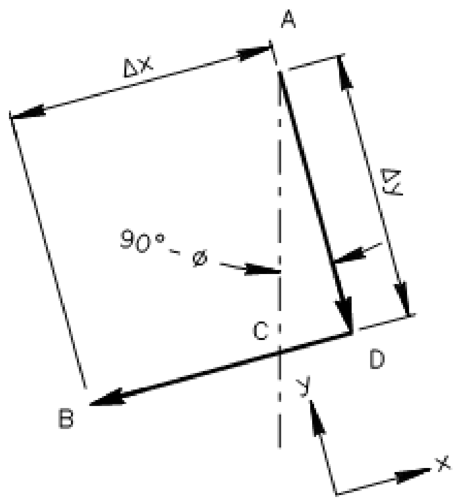
Check Δ_x versus max allowable of 1.5 inches using Load C:

Load in the x-direction W_x = 649 (sin (90-84.29)) = 64.57 lb/ft

$$\Delta_x = \frac{5WL^4}{384EI_{y-y}} = \frac{5\left(64.57\ ^{lb}/_{ft}\right)(48\ ft)^4\left(1728\ in^3/ft^3\right)}{384\ (30\ x\ 10^6\ psi)(838\ in^4)} = 0.31\ In.$$

Load in the y-direction W_y = 649(cos(90-84.29)) = 645.78 lb/ft

$$\Delta_y = \frac{5WL^4}{384EI_{x-x}} = \frac{5(645.78\ Lb/Ft)\ (48\ Ft)^4\left(1728\ In^3/Ft^3\right)}{384\ (30\ x\ 10^6\ psi)(2140\ In^4)} = 1.20\ in.$$



Lateral deflection = CB.

DC = AD [tan(90° - Ø)]

= 1.20 [tan(90° - 84.29°)]= 0.12 in.

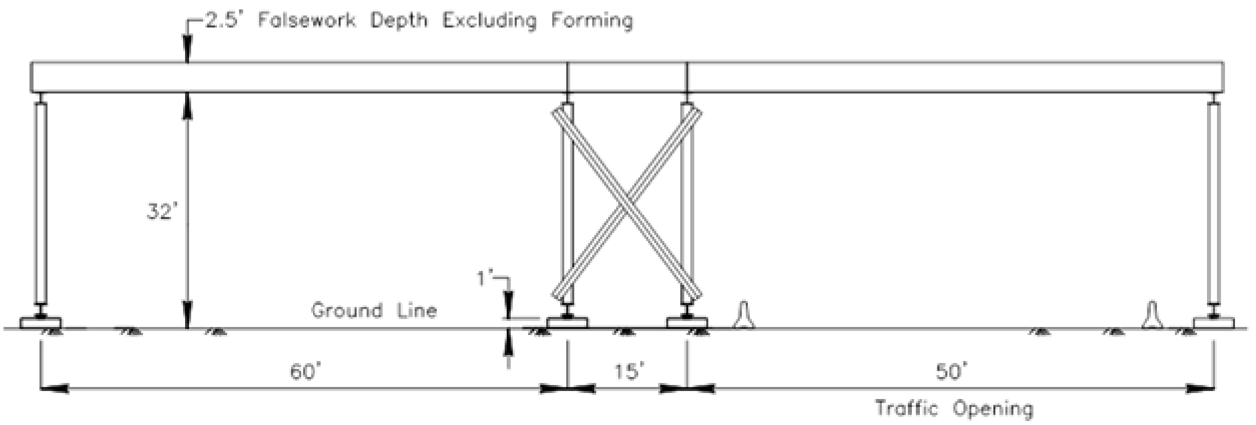
CB = DB - DC

= 0.31 - 0.12 = 0.19 < 1.5 in. **OK**

Appendix D Example 4 – Wind Loads on Conventional Falsework

This example demonstrates how to perform wind load calculations on conventional falsework. Refer to *Falsework Manual*, Section 3-3, *Horizontal Load.*

Given Information



Conventional Falsework

Transverse width of falsework = 58' (into paper)

Determine the Wind Load

Determine the Horizontal Design Wind Load for Bents A and B

1. Determine the width of the falsework system in the wind direction. W = 58'.
2. Calculate the drag coefficient Q.

   $Q = 1+0.2W = 1+0.2(58) = 12.6 > 10$

   $\therefore Q = 10$ max.

3. Calculate the wind pressure value for each height zone using the wind velocity coefficient for each height zone listed in *Standard Specifications*, Section 48-2.02B(2), *Falsework – Design Criteria – Loads:*

Height Zone	Bent A	Bent B
	All other locations	Adjacent to traffic locations
0-30	1.5Q = 1.5(10) = 15 psf	2.0Q = 2.0(10) = 20 psf
30-32	2.0Q = 2.0(10) = 20 psf	2.5Q = 2.5(10) = 25 psf
32-34.5	2.0Q = 2.0(10) = 20 psf	2.5Q = 2.5(10) = 25 psf

4. Calculate the wind impact area for each height zone:

Height Zone	Bent A	Bent B
0-30	30ft x $\frac{15\text{ ft}}{2}$ = 225 sqft	30ft x $\frac{15\text{ ft}}{2}$ = 225 sqft
30-32	2ft x $\frac{15\text{ ft}}{2}$ = 15 sqft	2ft x $\frac{15\text{ ft}}{2}$ = 15 sqft
32-34.5	2.5 ft x $\left(\frac{15\text{ ft}}{2} + \frac{60\text{ ft}}{2}\right)$ = 93.75 sqft	2.5ft x $\left(\frac{15\text{ ft}}{2} + \frac{50\text{ ft}}{2}\right)$ = 81.25 sqft

5. Calculate the total wind load for each height zone:

Height Zone	Bent A	Bent B
0-30	15 psf x 225 sqft = 3375 lb	20 psf x 225 sqft = 4500 lb
30-32	20 psf x 15 sqft = 300 lb	25 psf x 15 sqft = 375 lb
32-34.5	20 psf x 93.75 sqft = 1875 lb	25 psf x 81.25 sqft = 2031 lb

6. Calculate overturning moment.

Height Zone	Bent A	Bent B
0-30	3375 lb x 14ft = 47250 ft-lb	4500 lb x 14ft = 63000 ft-lb
30-32	300 lb x 30ft = 9000 ft-lb	375 lb x 30ft = 11250 ft-lb
32-34.5	1875 lb x 32.25ft = 60469 ft-lb	2031lb x 32.25ft = 65500 ft-lb
Total	116719 ft-lb	139750 ft-lb

7. Calculate the horizontal design wind load applied at top of post (bottom of top cap)

Height Zone	Bent A	Bent B
	$\frac{116719\ \text{ft}-\text{lb}}{31\text{ft}}$ =**3765 lbs**	$\frac{139750\text{ft}-\text{lb}}{31\text{ft}}$ = **4508 lbs**

Appendix D Example 5 – Horizontal Design Load on Heavy Duty Falsework

Refer to *Falsework Manual*, Section 3-3.03A *Wind Load on Heavy Duty Metal Shoring.* This example demonstrates how to calculate horizontal design loads on heavy duty falsework.

Given Information

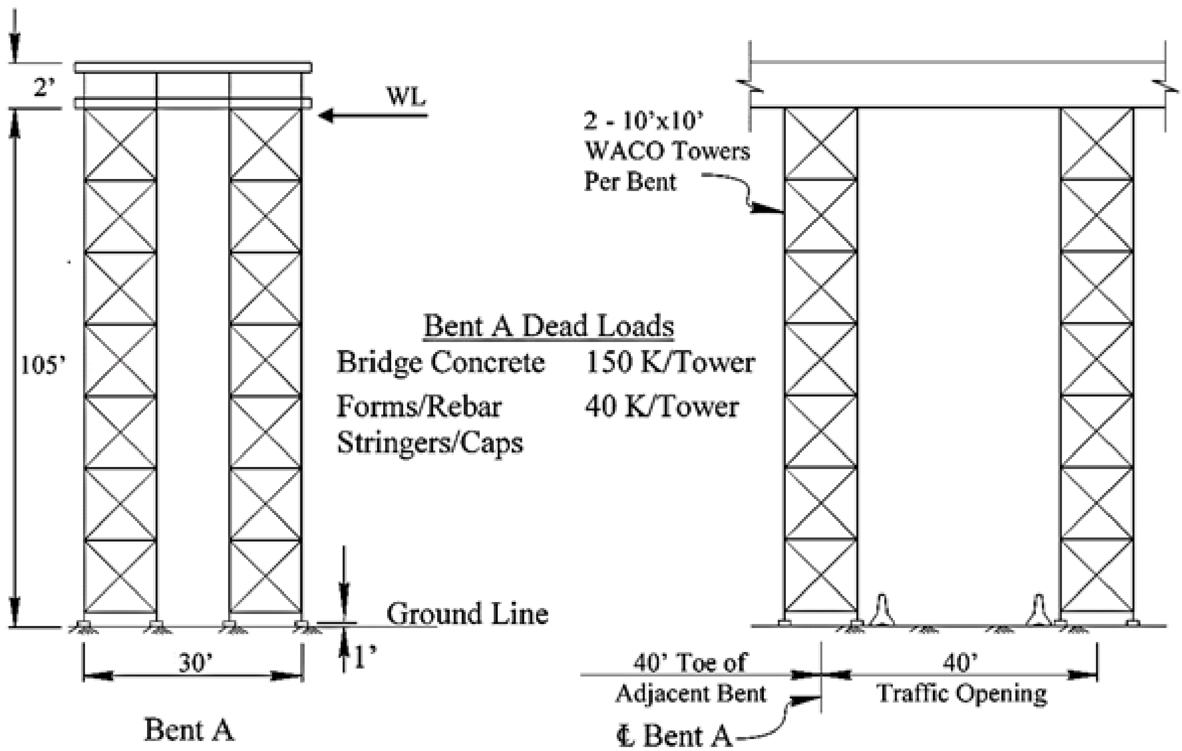


Figure D-5-1. Wind Load on Towers from Supported Falsework

Determine the Horizontal Design Load

Reference *Standard Specifications,* Section 48-2.02B(2), *Falsework – Design Criteria – Loads* , and *Falsework Manual,* Section 3-3, *Horizontal Load.*

Determine the Horizontal Wind Load (WL) for Bent A

1. Select the wind pressure values for each height zone above the ground from *Standard Specifications*, Section 48-2.02B(2), *Falsework – Design Criteria – Loads.*

2. Calculate the horizontal wind load for each height zone:
   - Shape factor for heavy-duty steel shoring = 2.2
   - Drag coefficient for conventional falsework $Q = 1+0.2W = 1+0.2(30) = 7 \leq 10$

     $\therefore Q = 7$

Height Zone (ft)	FW Type	Wind pressure (psf)	Horizontal wind load (psf)
0 - 30	Heavy Duty	20	20 x 2.2 = 44
30-50	″	25	25 x 2.2 = 55
50-100	″	30	30 x 2.2 = 66
100-105	″	35	35 x 2.2 = 77
105-107	Conventional	3.5Q	3.5(7) = 24.5

3. Calculate the total wind load <u>per tower</u> for each height zone:

   For tower section, full WL is applied to each tower

   *For supported FW section, 0.5 WL is applied to each tower

   For Waco shoring, 2 $^{\text{legs}}/_{\text{face}}$, use projected area = 2.0 $^{\text{sqft}}/_{\text{ft}}$

   $\Big\}$ See figures 3-4 thru 3-8 in sect. 3-3.03

4. Calculate the overturning moment about the base of the tower:

Height Zone (ft)	Horizontal wind load (psf)	Wind load (lbs)	Overturning moment (ft-lb)
0 - 30	44	44 x 2.00 x 30 = 2640	2640 x 14.5 = 38280
30-50	55	55 x 2.00 x 20 = 2200	2200 x 39 = 85800
50-100	66	66 x 2.00 x 50 = 6600	6600 x 74 = 488400
100-105	77	77 x 2.00 x 5 = 770	770 x 101.5= 78155
105-107	24.5	24.5 x 2 x 40' x 0.5* = 980	980 x 105 <u>= 102900</u>

Total overturning moment = 792215 ft-lb

5. Calculate the horizontal design wind load at the top of the tower:

$$\frac{793535\ \mathrm{ft-lb}}{104\ \mathrm{ft}} = 7630\ \mathrm{lb}$$

Determine the Horizontal Load from 2% Total Dead Load for Bent A

Dead load – bridge concrete $150\ {}^{\mathrm{Kip}}/_{\mathrm{Tower}}$

Dead load – forms/rebar/stringers/caps $\frac{40\ {}^{\mathrm{Kip}}/_{\mathrm{Tower}}}{190\ {}^{\mathrm{Kip}}/_{\mathrm{Tower}}}$

2% dead = 0.02 x 190k = 3.8k = 3800 lb

Determine the Controlling Horizontal Design Load for Bent A

$$\left.\begin{array}{l}\text{Wind load = 7630 lb}\\ \text{2\% dead load = 3800 lb}\end{array}\right\}\ \textbf{Horizontal Design Load = 7630 lb}$$

Appendix D Example 6 – Stability of Conventional Falsework in Longitudinal Direction

Refer to *Falsework Manual*, Section 6-4, *Longitudinal Stability.* This example demonstrates how to check stability of conventional falsework with horizontal forces applied in the longitudinal direction.

Given Information

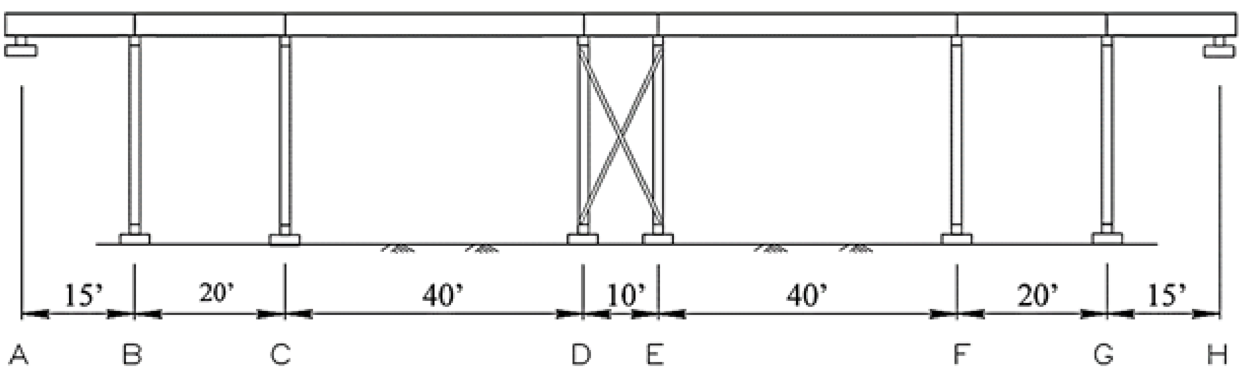


Figure D-6-1. Braced Falsework System

DL_1 = Weight of concrete per girder based on 160 pcf = 2000 plf

DL_2 = Weight of falsework stringer = 100 plf

The controlling horizontal force is 2% dead load.

Determine the Stability of the Bents

Investigate the stability of the falsework bents when the horizontal design load is applied in the longitudinal direction.

Calculate the Horizontal Design Load:

Span	Horizontal Design Load	Span	Horizontal Design Force
AB = GH	0.02 (2000 + 100) 15 = 630 lb	CD = EF	0.02(2000 + 100)40 = 1680 lb
BC = FG	0.02 (2000 + 100) 20 = 840 lb	DE	0.02(2000 + 100)10 = 420 lb

Calculate the Friction Transfer Capability (FTC)

From *Falsework (FW) Manual,* Section 6-4.03, *Friction,* FTC in the unloaded condition is the FTC that will be developed by the dead load of the falsework members plus an allowance for the weight of forms and reinforcing steel.

Weight of falsework members = 100 PLF

Weight of forms and reinforcing steel $\frac{15}{160}$ (2000 plf) = 188 plf

μ_S = 0.30 (*FW Manual*, Sect. 4-7.01)

Location			Location		
Between Bent	And Stringer	FTC (lb)	Between Bent	And Stringer	FTC (lb)
A B G H	AB BA GH HG	0.30(100+188) $\frac{15\text{ft}}{2} = 648$	C D E F	CD DC EF FE	0.30(100+188)$\frac{40\text{ft}}{2}$ = 1728
B C F G	BC CB FG GF	0.30(100+188) $\frac{20\text{ft}}{2} = 864$	D E	DE ED	0.30(100+188)$\frac{10\text{ft}}{2}$ = 432

As per *FW Manual*, Sect. 6-2, *Inherent Stability*:

Bents A & H are inherently stable (since post height < 3 times post width) and bracing is not required. Bents B, C, D, E, F, & G are not inherently stable (since post height > 3 times post width) and bracing, blocking, ties, etc. are required.

In the case of this example problem, bents D & E are made stable by diagonal bracing and bents B, C, F, & G will have to be made stable by strutting the horizontal forces to the stable bents.

Note: Strutting to stable bents can be forward or back and multiple load path solutions are possible. This example assumes loads will move forward.

Bent A

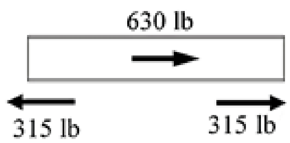


Figure D-6-2. Bent A Horizontal Forces

Half the horizontal design force for stringer AB is taken at stable bent A. Since the 315 lb is less than the FTC = 648 lb, no mechanical connection is required. The other half is strutted ahead through bent B.

Bent B

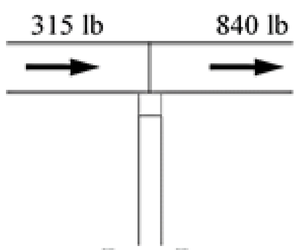


Figure D-6-3. Bent B Horizontal Forces

Since the FTC between bent B and stringer BA = 648 lb > 315 lb, and since the FTC between bent B and stringer BC = 864 lb > 315 lb, the 315 lb coming from span BA can be strutted ahead to a stable bent and no mechanical connections are required.

Bent C

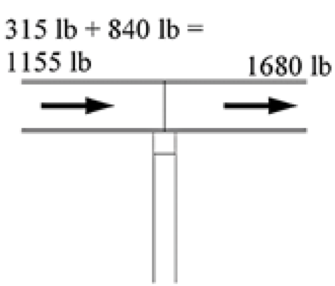


Figure D-6-4. Bent C Horizontal Forces

Since the FTC between bent C and Stringer CB = 864 lb < 1155 lb, and the FTC between bent C and stringer CD = 1728 lb >1155 lb, a mechanical connection between bent C and stringer CB will be required, but friction between bent C and stringer CD will be adequate to strut the 1155 lb coming from spans AB & BC to a stable bent.

Bent D

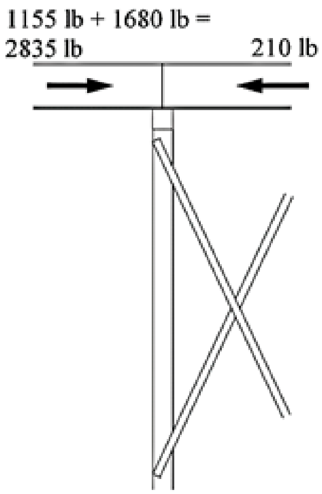


Figure D-6-5. Bent D Horizontal Forces

Since the FTC between bent D and stringer DC = 1728 lb < 2835 lb, a mechanical connection between bent D and stringer DC will be required.

The 420 lb in span DE will cause a reaction of 210 lb (at each end) which is < FTC between bent D and stringer DE = 432 lbs. Therefore, the 210 lb at bent D can be transferred to this stable bent by friction. The diagonal bracing at bent D then must take 2835 + 210 = 3045 lbs.

Bent E

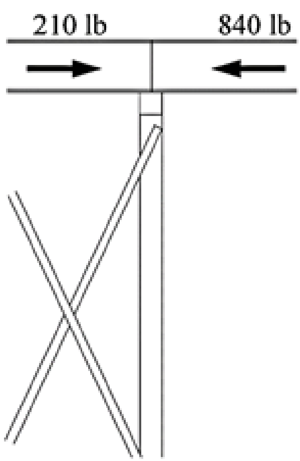
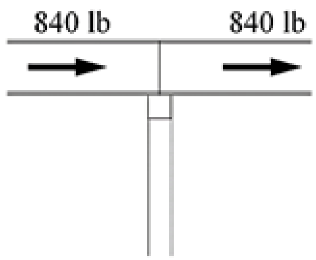


Figure D-6-6. Bent E Horizontal Forces

Since the FTC between bent E and stringer ED = 432 lb > 210lbs, the 210 lb can be taken to the stable bent E through friction. The 1680 lb in span EF will cause a reaction of 840 lb (at each end) and since the FTC between bent E and stringer EF = 1728 lb > 840lbs, the 840 lb can be taken to the stable bent E through friction and the diagonal bracing will then have to take 210 + 840 = 1050 lbs.

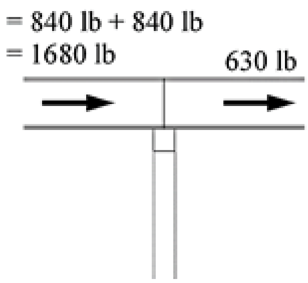
Bent F



Since the FTC between bent F and stringer FE = 1728 lb > 840 lb, and since the FTC between bent F and stringer FG = 864 lb > 840 lb, the 840 lb coming from span FE can be strutted ahead to a stable bent and no mechanical connections are required.

Figure D-6-7. Bent F Horizontal Forces

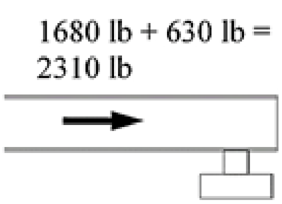
Bent G



Since the FTC between bent G and stringer GF = 864 lb < 1680 lb, and since the FTC between bent G and stringer GH = 648 lb < 1680 lb, mechanical connections between both stringers (GF and GH) and bent G will be required to strut the forces to stable bent H.

Figure D-6-8. Bent G Horizontal Forces

Bent H



Since the FTC between bent H and stringer HG = 648 lb < 2310 lb, a mechanical connection is required to get the forces coming from spans EF, FG, & GH into the stable bent H.

Figure D-6-9. Bent H Horizontal Forces

The diagonal bracing for bents D & E must be capable of resisting a total horizontal force of 3045 lb from bent D and 1050 lb from bent E provided the bents D and E are strutted so they act together. The total force acting on the strutted bracing system is 3045+1050 = 4095 lbs.

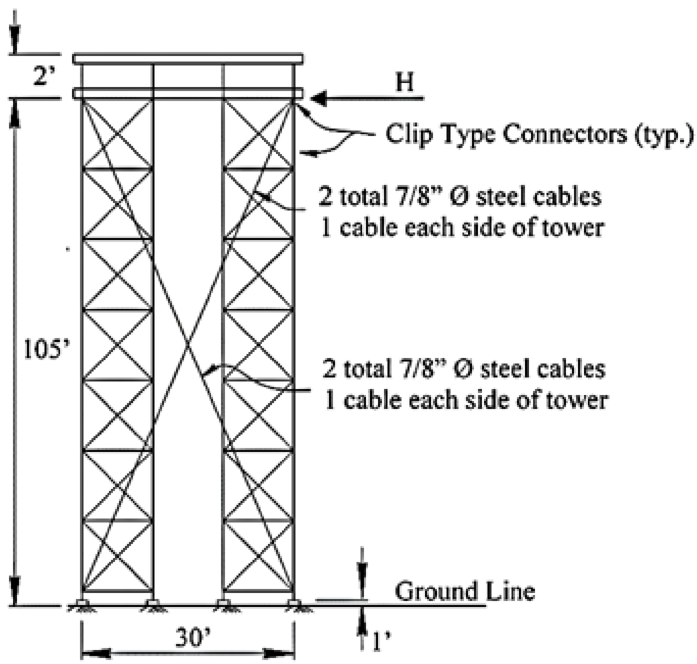
A similar analysis is required when the horizontal design forces are applied in the opposite direction.

Appendix D Example 7 – Stability of Shoring Towers

Refer to *Falsework (FW) Manual*, Section 6-6, *Tower Stability*. This example demonstrates how to check the stability of shoring towers.

Given Information

Wind load calculated in example 5:



H = 7630 lb per tower
(Wind Load from Example 5)

Ultimate cable load specified by manufacturer = 69.2 K

WACO 10x10 Towers

Weight of tower = $0.2 \frac{kip}{ft}$

Figure D-7-1. Wind Load on Towers

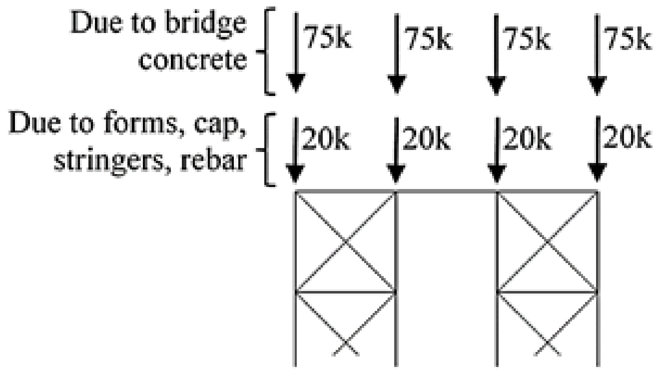


Figure D-7-2. Tower Reactions

Check Stability

Check Stability of the Unloaded Towers

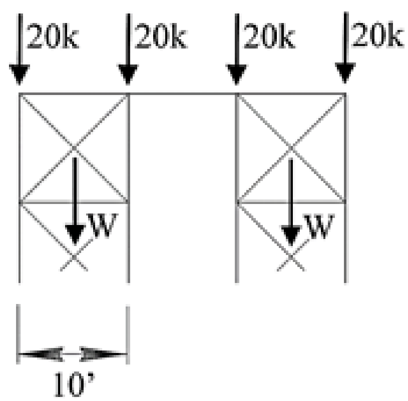


Figure D-7-3. Unloaded Towers

1. Calculate the resisting moment (RM) before the bridge concrete is placed:

   Tower weight W = 0.2 $\frac{\text{kip}}{\text{ft}}\ (104\ \text{ft}) = 20.8\ \text{kip}$

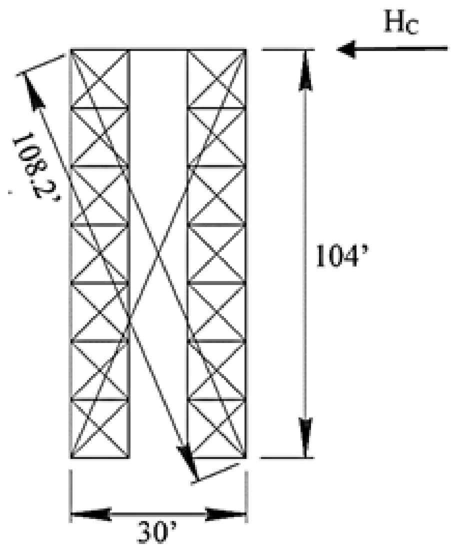
   RM per tower = 10 ft (20 k) + 5 ft (20.8 k) = $304^{\text{ft}-\text{k}}$

2. Overturning moment (OTM) = 104 ft (7630 lb/1000) = $794^{\text{ft}-\text{k}}$

3. Since OTM = $794^{\text{ft}-\text{k}} >$ RM = $304^{\text{ft}-\text{k}}$

   Cable bracing is required for unloaded condition.

4. Calculate the force in the cables.



Excess overturning, one tower = $794^{\text{ft}-\text{k}} - 304^{\text{ft}-\text{k}} = 490^{\text{ft}-\text{k}}$

Excess overturning, two towers = $2(490^{\text{ft}-\text{k}}) = 980^{\text{ft}-\text{k}}$

$H_C = \dfrac{980^{\text{ft}-\text{k}}}{104\text{ft}} = 9.42$ kip

Force in cables T = $\dfrac{108.2 \text{ ft}}{30 \text{ ft}}$ (9.42 kip) = 34.0 kip

Figure D-7-4. Towers and Cable Bracing

5. Check cables:

Efficiency of clip type connectors = 80% (FW Sect. 5-5.04)
Factor of safety = 3.0 (FW Sect. 5-5.06)

Ultimate cable load = 69.2 kip (given)

Safe working load –

$$\frac{\text{Breaking strength x connector efficiency}}{\text{safety factor}} = \frac{69.2 \text{ kip } (0.80)}{3.0} = 18.5 \text{ kip}$$

$18.5 \dfrac{\text{kip}}{\text{cable}} \text{ x } 2 \text{ cables} = 37.0 \text{ kip}$

34.0 kip ≤ 37.0 kip allowable

Check Stability of the Loaded Towers

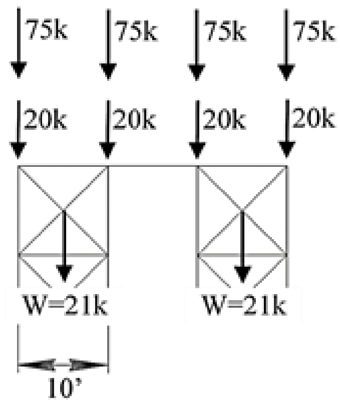


Figure D-7-5. Loaded Towers

1. Calculate the resisting moment after the bridge concrete is placed.

   RM of two tower units = 2 [5ft (21 kip)+ 10 ft (95 kip)] = $2110^{\text{ft}-\text{k}}$

2. OTM of two tower units = 2 x $794^{\text{ft}-\text{k}}$ = $1588^{\text{ft}-\text{k}}$

3. Since OTM =$1588^{\text{ft}-\text{k}}$ **≤** RM = $2110^{\text{ft}-\text{k}}$

   <u>Cable bracing is not required for loaded condition</u>

Appendix D Example 8 – Stability of Towers with Discontinuous Legs

Refer to *Falsework Manual,* Section 6, *Stability* and Section 6-6, *Tower Stability*. This example problem illustrates the overturning stability of a tower with discontinuous legs. Refer to Figure D-8-1.

Given Information

Assume that the bracing and other falsework features are adequate:

P_1 = 6,700 lb

P_2 = 7,000 lb

γ_ω = weight of wood = 35 pcf

H = 1,050 lb acting on one-half of a tower unit.

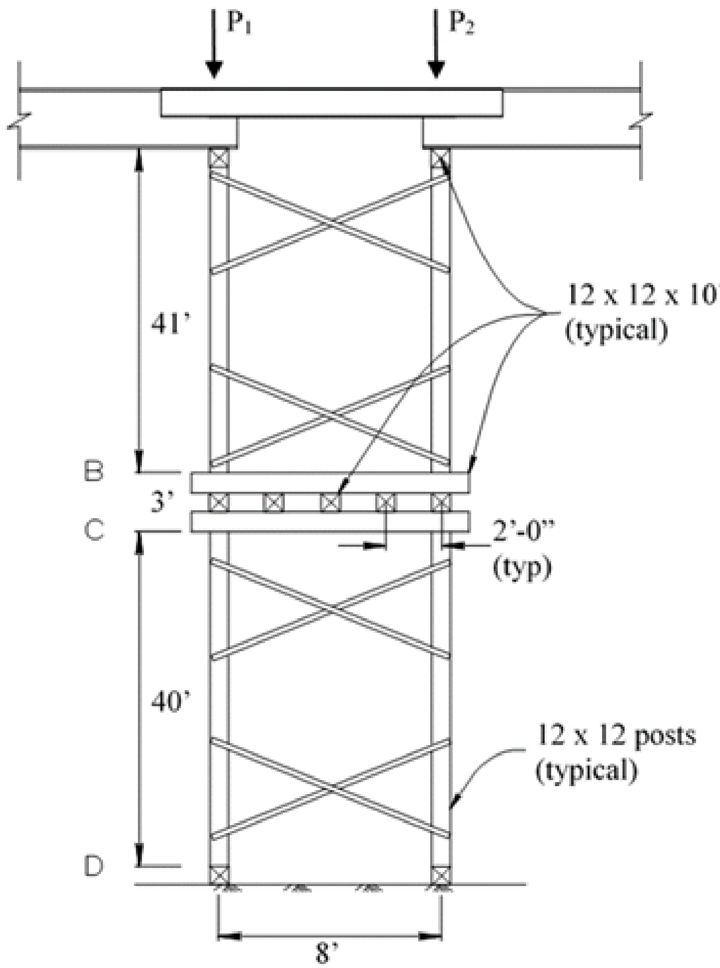


Figure D-8-1. Tower with Discontinuous Legs

Check Shear Resistance

The shear at the elevation of the plane B discontinuity will govern since frictional resistance increases with the weight of additional material below that elevation.

Check shear resistance at plane B. The active horizontal load (H) of 1,050 lb will be resisted by the frictional capacity of 2 tower legs.

Single post weight = 40 ft $(1\ \mathrm{ft}^2)$(35pcf) = 1,400 lb

Single cap weight = 10 ft $(1\ \mathrm{ft}^2)$(35pcf) = 350 lb

Resistance = 0.3[6,700 lb + 7,000 lb+ 2(1,400 lb) + 2$\left(\frac{350\ \mathrm{lb}}{2}\right)$] = 5,055 lb > 1,050 lb

Mechanical connection <u>not required</u>

Check Overturning Resistance

Check overturning resistance at plane B, C, and D by taking moments about the heavier loaded post.

Plane B:

OTM = 41 ft(H) = 41 ft(1,050 lb) = $43{,}050^{\text{ft}-\text{lb}}$

RM = 8ft (6,700 lb) + 8ft (1,400 lb) + 8 ft$\left(\frac{350\text{ lb}}{2}\right)$ = $66{,}200^{\text{ft}-\text{lb}}$

Safety Factor = $\frac{66{,}200}{43{,}050}$ = 1.54

External bracing not required

Plane C:

OTM = 44 ft(H) = 44 ft(1,050 lb) = $46{,}200^{\text{ft}-\text{lb}}$

RM = $66{,}200^{\text{ft}-\text{lb}}$ + 2(4 ft)(350 lb) + $\left(\frac{350\text{ lb}}{2}\right)$(2 ft + 4 ft + 6 ft + 8 ft) = $72{,}500^{\text{ft}-\text{lb}}$

Safety Factor = $\frac{72{,}500}{46{,}200}$ = 1.57

External bracing not required

Plane D:

OTM = 84 ft(H) = 84 ft(1,050 lb) = $88{,}200^{\text{ft}-\text{lb}}$

RM = $72{,}500^{\text{ft}-\text{lb}}$ + 8ft (1,400 lb)) = $83{,}700^{\text{ft}-\text{lb}}$

Safety Factor = $\frac{83{,}700}{88{,}200}$ = 0.95 **<** 1

External bracing will be required to prevent overturning!

Appendix D Example 9 – Effect of Overturning on Post Loads

Referring to Figure D-9-1, *Falsework Reactions*, in the unloaded and loaded condition the theoretical post load (dead load plus live load) will be increased or decreased by the post reaction created by the overturning moment, or the vertical component of the resisting couple acting through the post.

Unloaded Condition: Initial post load = 5 kips

Assumed horizontal load = 2 kips

Overturning about the bottom of the right leg

Loaded Condition: Initial post load = 50 kips

Assumed horizontal load = 2 kips

Overturning about the bottom of the right leg

$$\text{Vertical component due to overturning} = \frac{2\ \text{kip}\ (20\ \text{ft})}{10\ \text{ft}} = 4\ \text{kip}$$

In the bent shown the post design load is 54 kips.

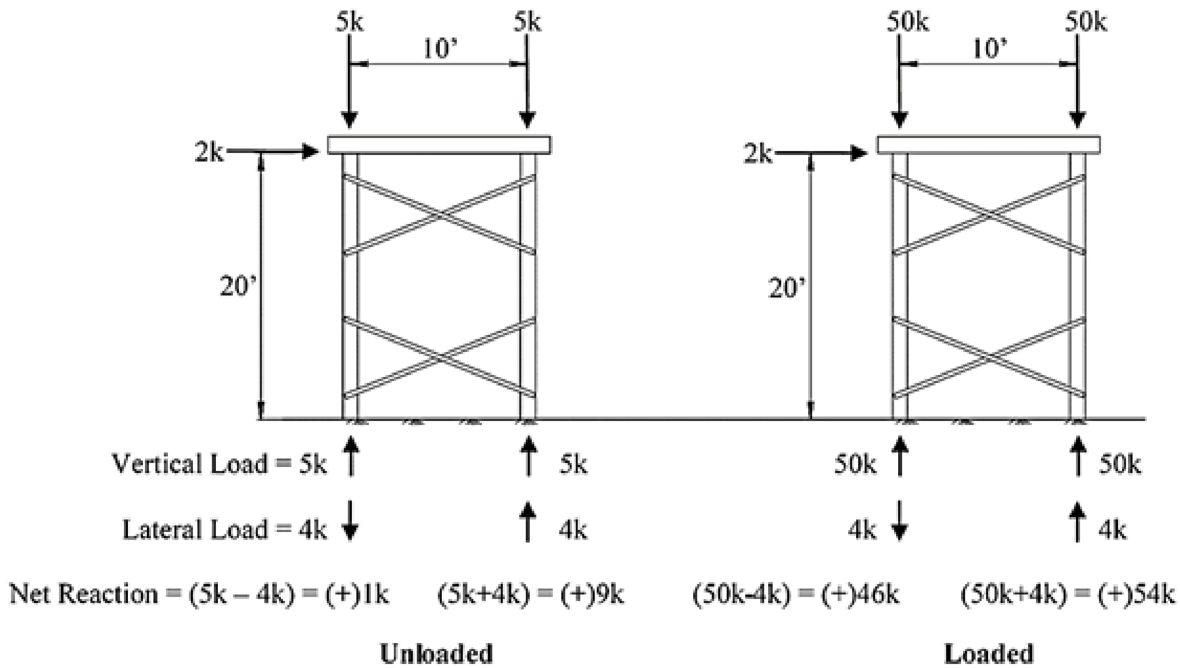
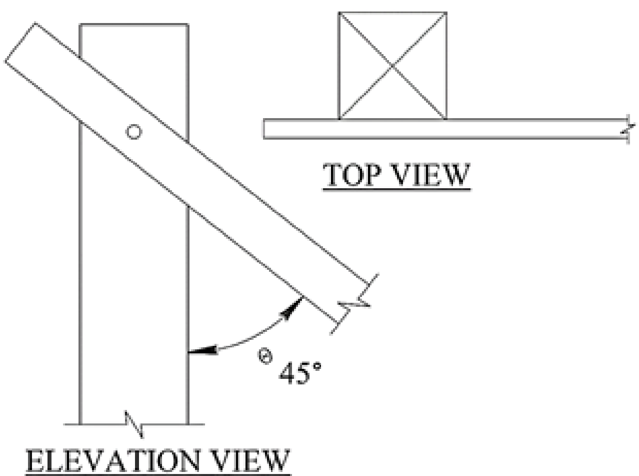


Figure D-9-1. Falsework Reactions

In a stable bent with more than two falsework posts, the post reactions are proportional to their distances from the center of rotation and may be obtained by algebraic summation.

Appendix D Example 10 – Bolted Joints - Single Shear

Refer to *Falsework Manual,* Section 5-3, *Timber Fasteners*. This example demonstrates how to calculate the capacity of the connection between a diagonal brace and post. For this example, wind load is the governing load.



Given Information

Posts:
12 x 12 Rough Douglas Fir-Larch #1 (G=0.50)

Diagonal Braces:
2 x 8 S4S Douglas Fir-Larch #2 (G=0.50)

Connectors:
¾" Ø Bolt

Figure D-10-1. Post and Brace Bolted Joint

Determine the connection capacity between brace and post for Wind Load

Main Member Properties

l_m = 12 in	*thickness (12 x 12)*
$t_m = l_m$ = 12 in	
θ_m= 45°	*angle between direction of loading & direction of grain*
G = 0.50	*Specific Gravity NDS Table 12.3.3*

Side Member Properties

l_s = 1.5 in	*thickness (2 x 8)*
$t_s = l_s$ = 1.5 in	
θ_s= 0°	*angle between direction of loading & direction of grain*

Connector Properties

$D = 0.75\ in$	*connector diameter*
$F_{yb} = 45000\ psi$	*Yield Strength (See Footnote #2 NDS Table 12A)*
$F_{e.pll} = 11200G\ psi = 5600\ psi$	*Dowel Bearing Strength Parallel to Grain (NDS table 12.3.3 footnote 2)*
$F_{e.perp} = \frac{6100G^{1.45}}{\sqrt{\frac{D}{in}}} = 2578\ psi$	*Dowel Bearing Strength Perpendicular to Grain (NDS table 12.3.3 footnote 2)*

Compare values to NDS Table 12.3.3:

$F_{e.pll\ (NDS\ Table\ 12.3.3)} = 5600\ psi$

$F_{e.perp\ (NDS\ Table\ 12.3.3)} = 2600\ psi$

Use calculated value for $F_{perp} = 2578\ psi$

Find Dowel Bearing Strength at an Angle to Grain (NDS Section 12.3.4):

$$F_{em} = \frac{F_{e.pll}F_{perp}}{F_{e.pll}(\sin(\theta_m))^2 + F_{perp}(\cos(\theta_m))^2} = 3531\ psi$$

$$F_{es} = \frac{F_{e.pll}F_{perp}}{F_{e.pll}(\sin(\theta_s))^2 + F_{perp}(\cos(\theta_s))^2} = 5600\ psi$$

Find Reduction Term, R_d (NDS Table 12.3.1B):

$\theta = \max(\theta_m, \theta_s) = 45°$	*Maximum angle between direction of load and direction of grain for any member in connection (See Table 12.3.1B)*
$K_\theta = 1 + 0.25\frac{\theta}{90\ deg} = 1.125$	
$R_{d\_I} = 4\ K_\theta = 4.50$	*Reduction Term for Yield Mode I_m and I_s*
$R_{d\_II} = 3.6\ K_\theta = 4.05$	*Reduction Term for Yield Mode II*
$R_{d\_III.IV} = 3.2\ K_\theta = 3.60$	*Reduction Term for Yield Mode III_m, III_s, and IV*

Find Yield Limit Equations for Single Shear (NDS Table 12.3.1A):

$$R_e = \frac{F_{em}}{F_{es}} = 0.6305$$

$$R_t = \frac{l_m}{l_s} = 8$$

$$k_1 = \frac{\sqrt{R_e + 2R_e^2(1 + R_t + R_t^2) + R_t^2 R_e^3} - R_e(1 + R_t)}{(1 + R_e)} = 1.8210$$

$$k_2 = -1 + \sqrt{2(1 + R_e) + \frac{2F_{yb}(1 + 2R_e)D^2}{3F_{em}l_m^2}} = 0.8265$$

$$k_3 = -1 + \sqrt{\frac{2(1 + R_e)}{R_e} + \frac{2F_{yb}(2 + R_e)D^2}{3F_{em}l_s^2}} = 2.2801$$

$$Z_{Im} = \frac{Dl_m F_{em}}{R_{d\_I}} = 7062 \text{ lb}$$ *NDS Eqn 12.3-1*

$$Z_{Is} = \frac{Dl_s F_{es}}{R_{d\_I}} = 1400 \text{ lb}$$ *NDS Eqn 12.3-2*

$$Z_{II} = \frac{k_1 Dl_s F_{es}}{R_{d\_II}} = 2833 \text{ lb}$$ *NDS Eqn 12.3-3*

$$Z_{IIIm} = \frac{k_2 Dl_m F_{em}}{(1 + 2R_e)R_{d\_III.IV}} = 3227 \text{ lb}$$ *NDS Eqn 12.3-4*

$$Z_{IIIs} = \frac{k_3 Dl_s F_{em}}{(2 + R_e)R_{d\_III.IV}} = 956 \text{ lb}$$ *NDS Eqn 12.3-5*

$$Z_{IV} = \frac{D^2}{R_{d\_III.IV}}\sqrt{\frac{2F_{em}F_{yb}}{3(1 + R_e)}} = 1259 \text{ lb}$$ *NDS Eqn 12.3-6*

The controlling value is the minimum single shear capacity from the above equations.

$Z_{control} = \min(Z_{Im}, Z_{Is}, Z_{II}, Z_{IIIm}, Z_{IIIs}, Z_{IV}) = 956$ lb (Yield Mode IIIs controls)

Find Adjusted Lateral Design Value, Z':

Adjustment factors from NDS Table 11.3.1:

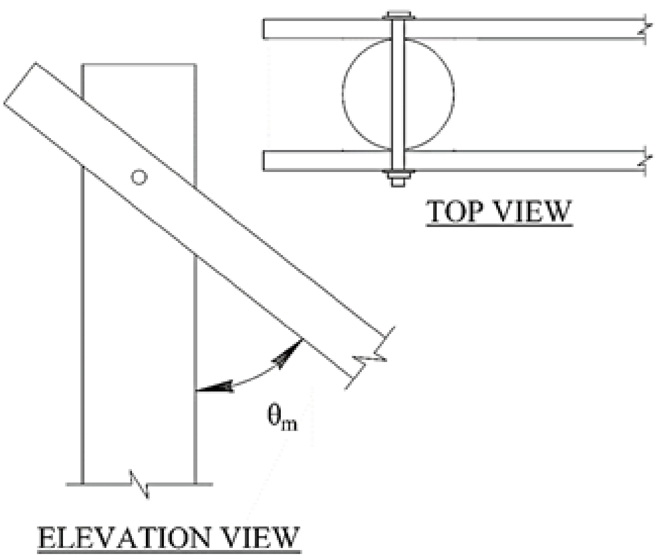
C_D = 1.60	*Duration Factor for wind load*
C_M = 1.0	*Wet Service Factor NDS 11.3.3 (Assume < 19% moisture content)*
C_t = 1.0	*Temperature Factor NDS 11.3.4 (Temp up to 100ºF)*
C_g = 1.0	*Group Action Factor NDS 11.3.6 (Single Fastener)*
C_Δ = 1.0	*Geometry Factor NDS 12.5.1 (Assume End Dist. & Spacing meet NDS Tables 12.5.1A and 12.5.1B)*
C_{eg} = 1.0	*End Grain Factor NDS 12.5.2 (Does not apply)*
C_{di} = 1.0	*Diaphragm Factor NDS 12.5.3 (Does not apply)*
C_{tn} = 1.0	*Toe Nail Factor NDS 12.5.4 (Does not apply)*

Adjusted lateral design value Z' = Z(C_D)(C_M)(C_t)(C_g)(C_Δ) = 1530 lb

Appendix D Example 11 – Bolted Joints - Double Shear

Refer to *Falsework Manual,* Section 5-3, *Timber Fasteners*. This example demonstrates how to calculate the capacity of the connection between a double diagonal brace and post. For this example, 2 % dead load is the governing load.

Given Information



Posts:
12” Diameter Pole

Diagonal Braces:
2x8 S4S Douglas Fir-Larch #2
(G=0.50)

Connectors:
¾” Ø Bolt

Figure D-11-1. Post and Double Brace Bolted Joint

Determine the connection capacity between brace and post for 2% Dead Load

Main Member Properties		Side Member Properties	
d_{pole} = 12 in	*diameter*	l_s = 1.5 in	*thickness (2x8)*
$\theta_m = \tan^{-1}(\frac{4}{3}) = 53.13°$	*angle between direction of loading & direction of grain*	$t_s = l_s$ = 1.5 in	
G = 0.50	*Specific Gravity NDS Table 12.3.3*	θ_s= 0°	*angle between direction of loading & direction of grain*

Connector Properties

$D = 0.75$ in *connector diameter*

$F_{yb} = 45000$ psi *Yield Strength (NDS table 12A footnote 2)*

Find equivalent square section width of pole:

$$l_m = \sqrt{\pi\left(\frac{d_{pole}}{2}\right)^2} = 10.63 \text{ in } (\textit{NDS 3.7.3})$$

$t_m = l_m = 10.63$ in

Find Dowel Bearing Strength at an Angle to Grain (NDS Section 12.3.4):

$F_{e.pll} = 11200G$ psi $= 5600$ psi *Dowel Bearing Strength Parallel to Grain (NDS table 12.3.3 footnote 2)*

$$F_{e.perp} = \frac{6100G^{1.45}}{\sqrt{D}} = 2578 \text{ psi}$$

Dowel Bearing Strength Perpendicular to Grain (NDS table 12.3.3 footnote 2)

Compare values to NDS Table 12.3.3:

$F_{e.pll\ (NDS\ Table\ 12.3.3)} = 5600$ psi

$F_{e.perp\ (NDS\ Table\ 12.3.3)} = 2600$ psi

Use calculated value for $F_{perp} = 2578$ psi

Find Dowel Bearing Strength at an Angle to Grain (NDS Section 12.3.4):

$$F_{em} = \frac{F_{e.pll}F_{perp}}{F_{e.pll}(\sin(\theta_m))^2 + F_{perp}(\cos(\theta_m))^2} = 3200 \text{ psi}$$

$$F_{es} = \frac{F_{e.pll}F_{perp}}{F_{e.pll}(\sin(\theta_s))^2 + F_{perp}(\cos(\theta_s))^2} = 5600 \text{ psi}$$

Find Reduction Term, R_d (NDS Table 12.3.1B):

$\theta = \max(\theta_m, \theta_s) = 53.13°$ — *Maximum angle between direction of load and direction of grain for any member in connection (See Table 12.3.1B)*

$K_\theta = 1+0.25\frac{\theta}{90\text{ deg}} = 1.15$

$R_{d\_I} = 4\ K_\theta = 4.59$ — *Reduction Term for Yield Mode I_m and I_s*

$R_{d\_II} = 3.6\ K_\theta = 4.13$ — *Reduction Term for Yield Mode II*

$R_{d\_III.IV} = 3.2\ K_\theta = 3.67$ — *Reduction Term for Yield Mode III_m, III_s, and IV*

Find Yield Limit Equations for Single Shear (NDS Table 12.3.1A):

$R_e = \frac{F_{em}}{F_{es}} = 0.571$

$R_t = \frac{l_m}{l_s} = 7.09$

Note: Values for k_1 and k_2 not required for double shear

$$k_3 = -1 + \sqrt{\frac{2(1+R_e)}{R_e} + \frac{2F_{yb}(2+R_e)D^2}{3F_{em}l_s^2}} = 2.3951$$

$$Z_{Im} = \frac{Dl_mF_{em}}{R_{d\_I}} = 5558\text{ lb}$$ *NDS Eqn 12.3-7*

$$Z_{Is} = \frac{2Dl_sF_{es}}{R_{d\_I}} = 2745\text{ lb}$$ *NDS Eqn 12.3-8*

$$Z_{IIIs} = \frac{2k_3Dl_sF_{em}}{(2+R_e)R_{d\_III.IV}} = 1826\text{ lb}$$ *NDS Eqn 12.3-9*

$$Z_{IV} = \frac{2D^2}{R_{d\_III.IV}}\sqrt{\frac{2F_{em}F_{yb}}{3(1+R_e)}} = 2394\text{ lb}$$ *NDS Eqn 12.3-10*

The controlling value is the minimum double shear capacity from the above equations.

$Z_{control} = \min(Z_{Im}, Z_{Is}, Z_{IIIs}, Z_{IV}) = 1826\text{ lb}$ *(Yield Mode IIIs controls)*

Find Adjusted Lateral Design Value, Z':

Adjustment factors from NDS Table 11.3.1:

C_D = 1.25 *Duration Factor for 2% lateral loading*

C_M = 1.0 *Wet Service Factor NDS 11.3.3 (Assume < 19% moisture content)*

C_t = 1.0 *Temperature Factor NDS 11.3.4 (Temp up to 100ºF)*

C_g = 1.0 *Group Action Factor NDS 11.3.6 (Single Fastener)*

C_Δ = 1.0 *Geometry Factor NDS 12.5.1 (Assume End Dist. & spacing meet NDS Tables 12.5.1A and 12.5.1B)*

C_{eg} = 1.0 *End Grain Factor NDS 12.5.2 (Does not apply)*

C_{di} = 1.0 *Diaphragm Factor NDS 12.5.3 (Does not apply)*

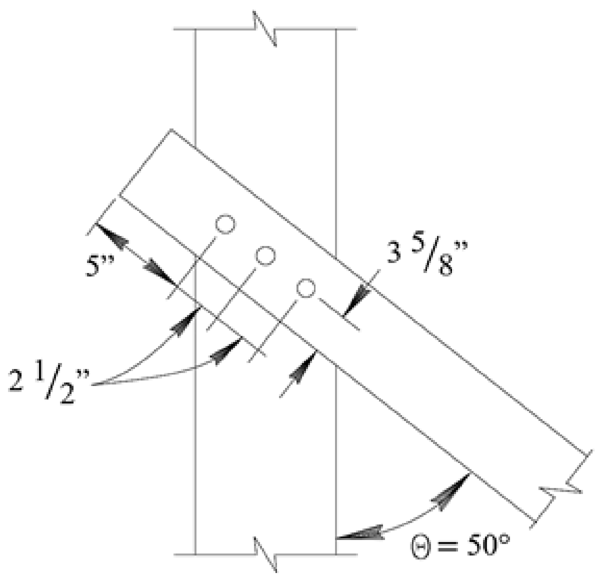
C_{tn} = 1.0 *Toe Nail Factor NDS 12.5.4 (Does not apply)*

Adjusted lateral design value Z' = $Z(C_D)(C_M)(C_t)(C_g)(C_\Delta)$ = 2283 lb

Appendix D Example 12 – Multiple Fastener Connection - Single Shear

Refer to *Falsework Manual,* Section 5-3, *Timber Fasteners*. This example demonstrates how to calculate the capacity of a multiple fastener connection between a single diagonal brace and post. For this example, 2 % dead load is the governing load.

Given Information



Posts:
12 x 12 Rough Douglas Fir-Larch #1 (G=0.50)

Diagonal Braces:
2x8 S4S Douglas Fir-Larch #2 (G=0.50)

Connectors:
Three 5/8" Ø Bolts in a single row
Center of gravity of the bolt group coincides with the center of gravity of the members.

Figure D-12-1. Post and Brace with Multiple Fasteners

Determine the connection capacity between brace and post for 2% Dead Load

Main Member Properties

$l_m = 12$ in	*thickness (12x12)*
$t_m = l_m = 12$ in	
$\theta_m = 50°$	*angle between direction of loading & direction of grain*
$E_m = 1600000$ psi	*modulus of elasticity NDS Table 4D*
$G = 0.50$	*Specific Gravity NDS Table 12.3.3*

Side Member Properties

$l_s = 1.5$ in	*thickness (2x8)*
$t_s = l_s = 1.5$ in	
$\theta_s = 0°$	*angle between direction of loading & direction of grain*
$E_s = 1600000$ psi	*Modulus of elasticity NDS Table 4A*

Connector Properties

$D = 0.625$ in	*connector diameter*
$n = 3$	*number of fasteners*
$F_{yb} = 45000$ psi	*Yield Strength (NDS table 12A, footnote 2)*
$F_{e.pll} = 11200G$ psi $= 5600$ psi	*Dowel Bearing Strength Parallel to Grain (NDS table 12.3.3 footnote 2)*

$$F_{e.perp} = \frac{6100G^{1.45}}{\sqrt{\frac{D}{in}}} = 2824 \text{ psi}$$

Dowel Bearing Strength Perpendicular to Grain (NDS table 12.3.3 footnote 2)

Compare values to NDS Table 12.3.3:

$F_{e.pll\,(\text{NDS Table 12.3.3})} = 5600$ psi

$F_{e.perp\,(\text{NDS Table 12.3.3})} = 2800$ psi

Use calculated value for $F_{perp} = 2824$ psi

Find Dowel Bearing Strength at an Angle to Grain (NDS Section 12.3.4):

$$F_{em} = \frac{F_{e.pll}F_{perp}}{F_{e.pll}(\sin(\theta_m))^2 + F_{perp}(\cos(\theta_m))^2} = 3551 \text{ psi}$$

$$F_{es} = \frac{F_{e.pll}F_{perp}}{F_{e.pll}(\sin(\theta_s))^2 + F_{perp}(\cos(\theta_s))^2} = 5600 \text{ psi}$$

Find Reduction Term, R_d (NDS Table 12.3.1B):

$\theta = \max(\theta_m, \theta_s) = 50°$ — *Maximum angle between direction of load and direction of grain for any member in connection (See Table 12.3.1B)*

$$K_\theta = 1 + 0.25 \frac{\theta}{90\ \text{deg}} = 1.139$$

$R_{d\_I} = 4\ K_\theta = 4.56$ — *Reduction Term for Yield Mode I_m and I_s*

$R_{d\_II} = 3.6\ K_\theta = 4.10$ — *Reduction Term for Yield Mode II*

$R_{d\_III.IV} = 3.2\ K_\theta = 3.64$ — *Reduction Term for Yield Mode III_m, III_s, and IV*

Find Yield Limit Equations for Single Shear (NDS Table 12.3.1A):

$$R_e = \frac{F_{em}}{F_{es}} = 0.6341$$

$$R_t = \frac{l_m}{l_s} = 8$$

$$k_1 = \frac{\sqrt{R_e + 2R_e^2\left(1 + R_t + R_t^2\right) + R_t^2 R_e^3} - R_e(1 + R_t)}{(1 + R_e)} = 1.8305$$

$$k_2 = -1 + \sqrt{2(1 + R_e) + \frac{2F_{yb}(1 + 2R_e)D^2}{3F_{em}l_m^2}} = 0.8221$$

$$k_3 = -1 + \sqrt{\frac{2(1 + R_e)}{R_e} + \frac{2F_{yb}(2 + R_e)D^2}{3F_{em}l_s^2}} = 2.0029$$

$$Z_{Im} = \frac{Dl_m F_{em}}{R_{d\_I}} = 5846\ \text{lb}$$ *NDS Eqn 12.3-1*

$$Z_{Is} = \frac{Dl_s F_{es}}{R_{d\_I}} = 1152\ \text{lb}$$ *NDS Eqn 12.3-2*

$$Z_{II} = \frac{k_1 D l_s F_{es}}{R_{d\_II}} = 2344\ \text{lb}$$ *NDS Eqn 12.3-3*

$$Z_{IIIm} = \frac{k_2 D l_m F_{em}}{(1 + 2R_e)R_{d\_III.IV}} = 2649\ \text{lb}$$ *NDS Eqn 12.3-4*

$$Z_{IIIs} = \frac{k_3 D l_s F_{em}}{(2 + R_e)R_{d\_III.IV}} = 695\ \text{lb}$$ *NDS Eqn 12.3-5*

$$Z_{IV} = \frac{D^2}{R_{d\_III.IV}}\sqrt{\frac{2F_{em}F_{yb}}{3(1 + R_e)}} = 865 \text{ lb}$$ *NDS Eqn 12.3-6*

The controlling value is the minimum single shear capacity from the above equations.

$Z_{control}$ = min (Z_{Im}, Z_{Is}, Z_{II}, Z_{IIIm}, Z_{IIIs}, Z_{IV}) = 695 lb *(Yield Mode IIIs controls)*

<u>Find Adjusted Lateral Design Value, Z':</u>

Adjustment factors from NDS Table 11.3.1:

C_D = 1.25	*Duration Factor for 2% lateral loading*
C_M = 1.0	*Wet Service Factor NDS 11.3.3 (Assume < 19% moisture content)*
C_t = 1.0	*Temperature Factor NDS 11.3.4 (Temp up to 100ºF)*
C_{eg} = 1.0	*End Grain Factor NDS 12.5.2 (Does not apply)*
C_{di} = 1.0	*Diaphragm Factor NDS 12.5.3 (Does not apply)*
C_{tn} = 1.0	*Toe Nail Factor NDS 12.5.4 (Does not apply)*

<u>Find the Group Action Factor C_g (NDS Section 11.3.6)</u>:

The Group Action Factor, Cg, accounts for load distribution within a fastener group.

$$C_g = \left[\frac{m(1 - m^{2n})}{n\left[(1 + R_{EA}m^n)(1 + m) - 1 + m^{2n}\right]}\right]\left(\frac{1 + R_{EA}}{1 - m}\right) = 0.98$$ *Group Action Factor NDS Eqn. 11.3-1*

where:

n	= 3	*Number of fasteners in a row*
A_m	$= t_m^2 = 144 \text{ in}^2$	*Area of post*
A_s	$= t_s$ x brace width = 10.88 in^2	*Area of brace*
E_m	= 1600000 psi	*Modulus of elasticity NDS Table 4D*
E_s	= 1600000 psi	*Modulus of elasticity NDS Table 4A*
R_{EA}	$= \min\left(\frac{E_s A_s}{E_m A_m}, \frac{E_m A_m}{E_s A_s}\right) = 0.08$	
D	= 0.625 in	*connector diameter*

γ $= 180000 \frac{\text{lb}}{\text{in}}\left(\frac{D}{\text{in}}\right)^{1.5} = 88939 \frac{\text{lb}}{\text{in}}$

Load/Slip modulus for connection Dowel-type fasteners in wood-to-wood connections

s_{bolt} $= \text{spacing}_{in.a.row\_actual} = 2.5 \text{ in}$

Center to center spacing between adjacent fasteners in a row

u $= 1 + \gamma \frac{s_{bolt}}{2}\left(\frac{1}{E_m A_m} + \frac{1}{E_s A_s}\right) = 1.007$

m $= u - \sqrt{u^2 - 1} = 0.888$

Find the Geometry Factor C_Δ (NDS Section 12.5.1):

The Geometry Factor, C_Δ, is based on the end distance, edge distance and spacing of the dowel-type fasteners. For $C_\Delta = 1.0$, the following requirements must be met:

1. End Distance Requirements (NDS Table 12.5.1A):

   For softwood (DF-L) with the force acting Parallel to Grain in Tension, for $C_{\Delta\ end} = 1.0$, the minimum end distance must be 7D.

   $dist_{end} = 7D = 7(0.625 \text{ in}) = 4.38 \text{ in}$
   $dist_{end\_actual} = 5 \text{ in}$

   $$dist_{end} < dist_{end\_actual} \therefore C_{\Delta\ end} = 1.0$$

   Note: If $dist_{end\_actual}$ was between the minimum end distances for $C_{\Delta\ end} = 0.5$ and 1.0, $C_{\Delta\ end}$ would be determined as follows:

   $$C_{\Delta\ end} = \frac{dist_{end\_actual}}{\text{minimum end distance for } C_{\Delta\ end}} = 1.0$$

2. Shear area requirements (NDS Section 12.5.1(b)):

   In this case, the dowel-type fastener is not being loaded at an angle as shown in NDS Figure 12E. Therefore, the shear area geometry factor is $C_{\Delta\ shear\_area} = 1.0$.

3. Spacing Requirements for Fasteners in a Row (NDS Table 12.5.1B):

   Similar to the end distance requirements, the brace member is loaded parallel to grain. According to NDS Table 12.5.1B, the minimum spacing between fasteners in a row for $C_{\Delta\ in.a.row} = 1.0$ is 4D.

$spacing_{in.a.row} = 4D = 4(0.625\text{ in}) = 2.5\text{ in}$

$spacing_{in.a.row\_actual} = 2.5\text{ in}$

$$spacing_{in.a.row} = spacing_{in.a.row\_actual} \therefore C_{\Delta\ in.a.row} = 1.0$$

4. Edge Distance Requirements (NDS Table 12.5.1(c)):
   The edge distance requirement is determined by $\frac{l_s}{D}$ or $\frac{l_m}{D}$, whichever is smaller. For this case, $\frac{l_s}{D}$ is the smaller ratio. For the parallel to grain loading on the brace:

   $\frac{l_s}{D} = 2.4 \leq 6 \rightarrow$ the minimum edge distance is 1.5D

   $dist_{edge} = 1.5D = 1.5(.625\text{ in}) = 0.94\text{ in}$

   $dist_{edge.\ \_actual} = 3.625\text{ in}$

$$dist_{edge} < dist_{edge\_actual} \therefore C_{\Delta\ edge} = 1.0$$

5. Spacing Requirements Between Rows (NDS Table 12.5.1D):
   Since there is only one row of bolts, this condition does not apply.

$$C_{\Delta\ row} = 1.0$$

The Geometry Factor is the minimum factor of all the conditions.

$C_\Delta = \min(C_{\Delta\ end}, C_{\Delta\ shear\_area}, C_{\Delta\ in.a.row}, C_{\Delta\ edge}, C_{\Delta\ row}) = 1.0$

Adjusted lateral design value Z'

$Z' = nZ_{control}(C_D)(C_M)(C_t)(C_g)(C_\Delta) = 3(695\text{ lb})(1.25)(1.0)(1.0)(0.98)(1.0) = \underline{2554\text{ lb}}$

Appendix D Example 13 – Multiple Fastener Connection – Double Shear

Refer to *Falsework Manual,* Section 5-3, *Timber Fasteners*. This example demonstrates how to calculate the capacity of a multiple fastener connection between a double diagonal brace and post. For this example, wind load is the governing load.

Given Information

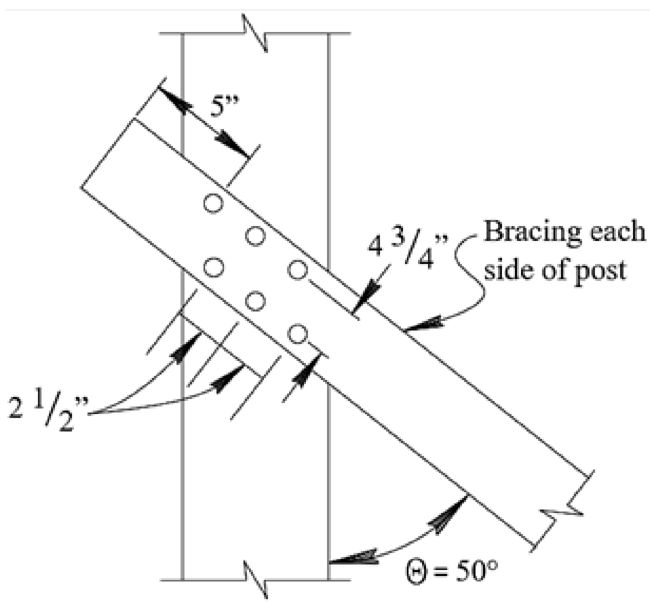


Figure D-13-1. Post and Double Brace with Multiple Fasteners

Posts:
12 x 12 Rough Douglas Fir-Larch #2
(G=0.50)

Diagonal Braces:
2x8 S4S Douglas Fir-Larch #2 each side
(G=0.50)

Connectors:
Six 5/8" Ø Bolts in two rows
Center of gravity of the bolt group coincides with the center of gravity of the members.

Assume:
Temperature Exposure up to 120°F

Determine the connection capacity between brace and post for Wind Load

Main Member Properties

$l_m = 12$ in	*thickness (12x12)*
$t_m = l_m = 12$ in	
$\theta_m = 50°$	*angle between a direction of loading & direction of grain*
$E_m = 1300000$ psi	*modulus of elasticity NDS Table 4D*
$G = 0.50$	*Specific Gravity NDS Table 12.3.3*

Side Member Properties

$l_s = 1.5$ in	*thickness (2x8)*
$t_s = l_s = 1.5$ in	
$\theta_s = 0°$	*angle between direction of loading & direction of grain*
$E_s = 1600000$ psi	*Modulus of elasticity NDS Table 4A*

Connector Properties

$D = 0.625$ in	*connector diameter*
$n = 3$	*number of fasteners per row*
$n_{rows} = 2$	*number of rows*
$F_{yb} = 45000$ psi	*Yield Strength (NDs table 12A footnote 2)*
$F_{e.pll} = 11200G \text{ psi} = 5600$ psi	*Dowel Bearing Strength Parallel to Grain (NDS table 12.3.3 footnote 2)*
$F_{e.perp} = \dfrac{6100G^{1.45}}{\sqrt{\dfrac{D}{in}}} = 2824$ psi	*Dowel Bearing Strength Perpendicular to Grain (NDS table 12.3.3 footnote 2)*

Compare values to NDS Table 12.3.3:

$F_{e.pll\ (NDS\ Table\ 12.3.3)} = 5600$ psi

$F_{e.perp\ (NDS\ Table\ 12.3.3)} = 2800$ psi

Use calculated value for $F_{perp} = 2824$ psi

Find Dowel Bearing Strength at an Angle to Grain (NDS Section 12.3.4):

$$F_{em} = \frac{F_{e.pll}F_{perp}}{F_{e.pll}(\sin(\theta_m))^2 + F_{perp}(\cos(\theta_m))^2} = 3551 \text{ psi}$$

$$F_{es} = \frac{F_{e.pll}F_{perp}}{F_{e.pll}(\sin(\theta_s))^2 + F_{perp}(\cos(\theta_s))^2} = 5600 \text{ psi}$$

Find Reduction Term, R_d (NDS Table 12.3.1B):

$\theta = \max(\theta_m, \theta_s) = 50°$ *Maximum angle between direction of load and direction of grain for any member in connection (See Table 12.3.1B)*

$K_\theta = 1 + 0.25\frac{\theta}{90\text{ deg}} = 1.14$

$R_{d\_I} = 4\ K_\theta = 4.56$ *Reduction Term for Yield Mode I_m and I_s*

$R_{d\_II} = 3.6\ K_\theta = 4.10$ *Reduction Term for Yield Mode II*

$R_{d\_III.IV} = 3.2\ K_\theta = 3.64$ *Reduction Term for Yield Mode III_m, III_s, and IV*

Find Yield Limit Equations for Single Shear (NDS Table 12.3.1A):

$R_e = \frac{F_{em}}{F_{es}} = 0.634$

$R_t = \frac{l_m}{l_s} = 8$

Note: Values for k_1 and k_2 not required for double shear

$$k_3 = -1 + \sqrt{\frac{2(1+R_e)}{R_e} + \frac{2F_{yb}(2+R_e)D^2}{3F_{em}l_s^2}} = 2.00$$

$$Z_{Im} = \frac{Dl_mF_{em}}{R_{d\_I}} = 5846 \text{ lb}$$ *NDS Eqn 12.3-7*

$$Z_{Is} = \frac{2Dl_sF_{es}}{R_{d\_I}} = 2305 \text{ lb}$$ *NDS Eqn 12.3-8*

$$Z_{IIIs} = \frac{2k_3Dl_sF_{em}}{(2+R_e)R_{d\_III.IV}} = 1389 \text{ lb}$$ *NDS Eqn 12.3-9*

$$Z_{IV} = \frac{2D^2}{R_{d\_III.IV}}\sqrt{\frac{2F_{em}F_{yb}}{3(1+R_e)}} = 1731 \text{ lb}$$ *NDS Eqn 12.3-10*

The controlling value is the minimum single shear capacity from the above equations.

$Z_{control} = \min(Z_{Im}, Z_{Is}, Z_{IIIs}, Z_{IV}) = 1389$ lb *(Yield Mode IIIs controls)*

Find Adjusted Lateral Design Value, Z':

Adjustment factors from NDS Table 11.3.1:

C_D = 1.6	*Duration Factor for wind load*
C_M = 1.0	*Wet Service Factor NDS 11.3.3 (Assume < 19% moisture content)*
C_t = 1.0	*Temperature Factor NDS 11.3.4 (Temp up to 120ºF)*
C_{eg} = 1.0	*End Grain Factor NDS 12.5.2 (Does not apply)*
C_{di} = 1.0	*Diaphragm Factor NDS 12.5.3 (Does not apply)*
C_{tn} = 1.0	*Toe Nail Factor NDS 12.5.4 (Does not apply)*

Find the Group Action Factor C_g (NDS Section 11.3.6):

The Group Action Factor, Cg, accounts for load distribution within a fastener group.

$$C_g = \left[\frac{m(1 - m^{2n})}{n\left[(1 + R_{EA}m^n)(1 + m) - 1 + m^{2n}\right]}\right]\left(\frac{1 + R_{EA}}{1 - m}\right) = 0.99$$

Group Action Factor NDS Eqn. 11.3-1

where:

n	= 3	*Number of fasteners in a row*
A_m	$= t_m^2 = 144\ in^2$	*Area of post*
A_s	$= 2 \times t_s \times$ brace width $= 21.75\ in^2$	*Area of brace*
E_m	= 1300000 psi	*Modulus of elasticity NDS Table 4D*
E_s	= 1600000 psi	*Modulus of elasticity NDS Table 4A*
R_{EA}	$= \min\left(\frac{E_s A_s}{E_m A_m}, \frac{E_m A_m}{E_s A_s}\right) = 0.19$	
D	= 0.625 in	*connector diameter*
γ	$= 180000\ \frac{lb}{in}\left(\frac{D}{in}\right)^{1.5} = 88939\ \frac{lb}{in}$	*Load/Slip modulus for connection Dowel-type fasteners in wood-to-wood connections*
s_{bolt}	$= \text{spacing}_{in.a.row\_actual} = 2.5\ in$	*Center to center spacing between adjacent fasteners in a row*
u	$= 1 + \gamma\,\frac{s_{bolt}}{2}\left(\frac{1}{E_m A_m} + \frac{1}{E_s A_s}\right) = 1.004$	
m	$= u - \sqrt{u^2 - 1} = 0.9145$	

Find the Geometry Factor C_Δ (NDS Section 12.5.1):

The Geometry Factor, C_Δ, is based on the end distance, edge distance and spacing of the dowel-type fasteners. To find if $C_\Delta = 1.0$, check for the following requirements:

1. End Distance Requirements (NDS Table 12.5.1A):

   For softwood (DF-L) with the force acting Parallel to Grain in Tension, for $C_{\Delta\ end} = 1.0$, the minimum end distance must be 7D.

   $$dist_{end} = 7D = 7(0.625\ in) = 4.38\ in$$
   $$dist_{end\_actual} = 5\ in$$

   $$dist_{end} < dist_{end\_actual} \therefore C_{\Delta\ end} = 1.0$$

   Note: If $dist_{end\_actual}$ was between the minimum end distances for $C_{\Delta\ end} = 0.5$ and 1.0, $C_{\Delta\ end}$ would be determined as follows:

   $$C_{\Delta\ end} = \frac{dist_{end\_actual}}{\text{minimum end distance for } C_{\Delta\ end} = 1.0}$$

2. Shear Area Requirements (NDS Section 12.5.1.2(b)):
   In this case, the dowel-type fastener is not being loaded at an angle as shown in NDS Figure 12E. Therefore, the shear area factor is $C_{\Delta\ shear\_area} = 1.0$.

   2018 National Design Specification (NDS) for Wood construction
   Figure 12E

   Note: Similar to End Distance, if shear area$_{actual}$ was between the minimum shear

3. Spacing Requirements for Fasteners in a Row (NDS Table 12.5.1B):

   Similar to the end distance requirements, the brace member is loaded parallel to grain. According to NDS Table 12.5.1B, the minimum spacing between fasteners in a row for $C_{\Delta\ in.a.row} = 1.0$ is 4D.

   $$spacing_{in.a.row} = 4D = 4(0.625\ in) = 2.5\ in$$
   $$spacing_{in.a.row\_actual} = 2.5\ in$$

   $$spacing_{in.a.row} = spacing_{in.a.row\_actual} \therefore C_{\Delta\ in.a.row} = 1.0$$

4. Edge Distance Requirements (NDS Table 12.5.1C):

   The edge distance requirement is determined by $\frac{l_s}{D}$ or $\frac{l_m}{D}$, whichever is smaller.

   For this case, $\frac{l_s}{D}$ is the smaller ratio. For the parallel to grain loading on the brace:

$\frac{l_s}{D} = 2.4 \le 6$ ➔ the minimum edge distance is 1.5D

$dist_{edge} = 1.5D = 1.5(.625\text{ in}) = 0.94\text{ in}$

$dist_{edge.\_actual} = 1\ 1/4\text{ in}$

$dist_{edge} < dist_{edge\_actual} \therefore C_{\Delta\ edge} = 1.0$

5. Spacing Requirements Between Rows (NDS Table 12.5.1D):

Similar to edge distance requirements, the ratio of $\frac{l_s}{D}$ is used to determine the minimum spacing between rows. For the parallel to grain loading on the brace, the minimum spacing is 1.5D.

$dist_{row} = 1.5D = 1.5(.625\text{ in}) = 0.94\text{ in}$

$dist_{row\_actual} = 4.75\text{ in}$

$dist_{row} < dist_{row\_actual} \therefore C_{\Delta\ row} = 1.0$

The Geometry Factor is the minimum factor of all the conditions.

$C_\Delta = \min(C_{\Delta\ end}, C_{\Delta\ shear\_area}, C_{\Delta\ in.a.row}, C_{\Delta\ edge}, C_{\Delta\ row}) = 1.0$

Adjusted lateral design value Z'

$Z' = (n_{rows})(n)Z_{control}(C_D)(C_M)(C_t)(C_g)(C_\Delta)$

$= (2\text{ rows})(3)(1389\text{ lb})(1.6)(1.0)(1.0)(0.99)(1.0) =$ **13201 lb**

Appendix D Example 14 – Diagonal Bracing of Single Tier Framed Bent – Nailed Connections

Refer to *Falsework Manual,* Section 6-3, *Diagonal Bracing* and Section 5-3, *Timber Fasteners*. This example demonstrates how to determine if the bracing system of a single tier framed bent is adequate. All connections are nailed.

Given Information

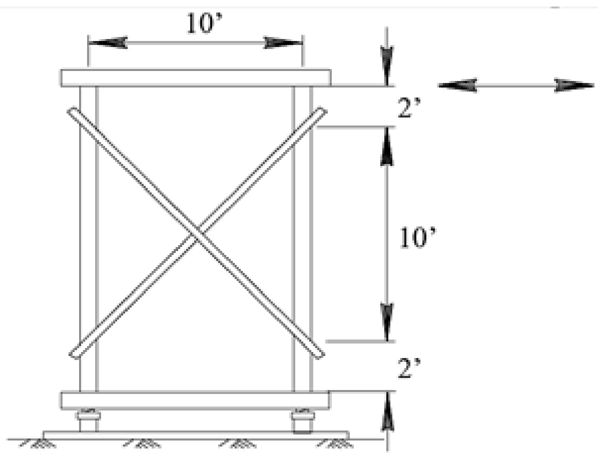


Figure D-14-1. Single Tier Framed Bent with Diagonal Bracing

2% Dead Load = 1900 lb
Wind Load = 1800 lb

Posts:
12 x 12 Rough Douglas Fir-Larch #1 (G=0.50)

Diagonal Braces:
2x8 Douglas Fir-Larch #2 (G=0.50)

Connectors:
Brace to Post 10-20d common nails
Intersection of Brace 4-16d common nails

Determine if the Bracing System is Adequate

1. **Determine the connection capacity between the brace and post:**

   10-20d Common Wire Nails

   Length = 4″

   Diameter (D) = 0.192″

   Penetration = 4″-1.5″ = 2.5″

   Minimum penetration for full tabular value = 10D = 10(0.192”) = 1.92”

   Minimum penetration = 6D = 6(0.192”) = 1.15”

   Reference lateral design value (Z) from NDS table 12N = 170 lb

Adjustment factors from NDS Table 11.3.1:

C_D = 1.25	*Duration Factor for 2% lateral loading*
C_M = 1.0	*Wet Service Factor NDS 11.3.3 (Assume < 19% moisture content)*
C_t = 1.0	*Temperature Factor NDS 11.3.4 (Temp up to 100ºF)*
C_g = 1.0	*Group Action Factor NDS 11.3.6*
C_Δ = 1.0	*Geometry Factor NDS 12.5.1*
C_{eg} = 1.0	*End Grain Factor NDS 12.5.2*
C_{di} = 1.0	*Diaphragm Factor NDS 12.5.3*
C_{tn} = 1.0	*Toe Nail Factor NDS 12.5.4*

Adjusted lateral design value $Z' = Z(C_D)(C_M)(C_t)(C_g)(C_\Delta)(C_{eg})(C_{di})(C_{tn})$ = 213 lb

Connection capacity = nZ' = 10(213 lb) = 2130 lb (n equals number of nails)

2. Determine the capacity of the diagonal brace in tension:

Reference design value in tension F_t = 575 psi (NDS supplement table 4A)

Adjustment factors from NDS table 4.3.1:

C_D = 1.25	*Duration Factor for 2% lateral loading*
C_M = 1.0	*Wet Service Factor NDS table 4A (Assume < 19% moisture content)*
C_t = 1.0	*Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)*
C_F = 1.2	*Size Factor NDS Table 4A*
C_i = 1.0	*Incising Factor NDS 4.3.8*

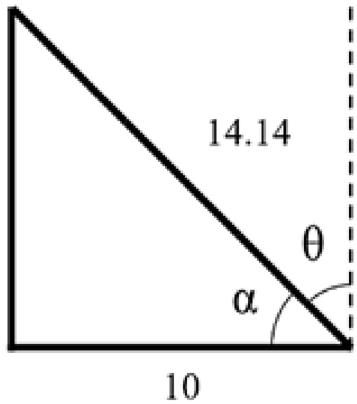
Adjusted design value $F_t' = F_t\,(C_D)(C_M)(C_t)(C_F)(C_i)$ = 863 psi

Tension capacity = 863 psi(1.5”)(7.25”) = 9385 lb

3. Determine the strength value of the tension members:

9385 lb > 2130 lb $\therefore$ Connection strength controls

4. Calculate the horizontal component of the strength value for the tension members



$$T = 2130\text{ lb}\left(\frac{10}{14.14}\right) = 1506\text{ lb}$$

Figure D-14-2. Geometric Components of Tension Strength Value

5. Determine the capacity of diagonal brace in compression:

First check adequacy of the connection to reduce the unsupported length of compression member (See section 6-3.02 Wood cross bracing):

4-16d Common Wire Nails

Reference withdrawal design value W = 40 lb/inch of penetration (NDS table 12.2C)

Adjustment factors from NDS Table 11.3.1:

C_D = 1.25 *Duration Factor for 2% lateral loading*
C_M = 1.0 *Wet Service Factor NDS 11.3.3 (Assume < 19% moisture content)*
C_t = 1.0 *Temperature Factor NDS 11.3.4 (Temp up to 100ºF)*
C_{eg} = 1.0 *End Grain Factor NDS 12.5.2*
C_{tn} = 1.0 *Geometry Factor NDS 12.5.4*

Adjusted withdrawal design value W' = $W(C_D)(C_M)(C_t)(C_{eg})(C_{tn})$ = 50 lb/inch

Penetration p = 1.5"

Connection capacity = n(p)(Z') = 4(1.5")(50 lb/inch) = 300 lb (n equals number of nails)

300 lb > 250 lb (minimum required per section 6-3.02)

Check cross brace capacity in compression:

Reference design value in compression F_c = 1350 psi (NDS supplement table 4A)

Adjustment factors from NDS table 4.3.1:

C_D = 1.25	*Duration Factor for 2% lateral loading*
C_M = 1.0	*Wet Service Factor NDS table 4A (Assume < 19% moisture content)*
C_t = 1.0	*Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)*
C_F = 1.05	*Size Factor NDS Table 4A*
C_i = 1.0	*Incising Factor NDS 4.3.8*
C_P = 0.083	*Column Stability Factor NDS 3.7.1 (unsupported length =* $\frac{14.14}{2}$= *7.07')*

Adjusted design compression value F_c' = F_c $(C_D)(C_M)(C_t)(C_F)(C_i)(C_P)$ = 147 psi

Compression brace capacity = 147 psi (1.5")(7.25") = 1599 lb

6. Determine the strength value of the compression members:

Connection capacity = 2130 lb

(See step 1. Capacity in tension and compression are the same)

1599 lb < 2130 lb ∴ 2x8 brace controls compression

Limit to ½ theoretical strength for compression values: See section 6-3.02, *Wood Cross Bracing.*

Reduced compression brace capacity = $\frac{1599 \text{ lb}}{2}$ = 800 lb

7. Calculate the horizontal component of the strength value for the compression member:

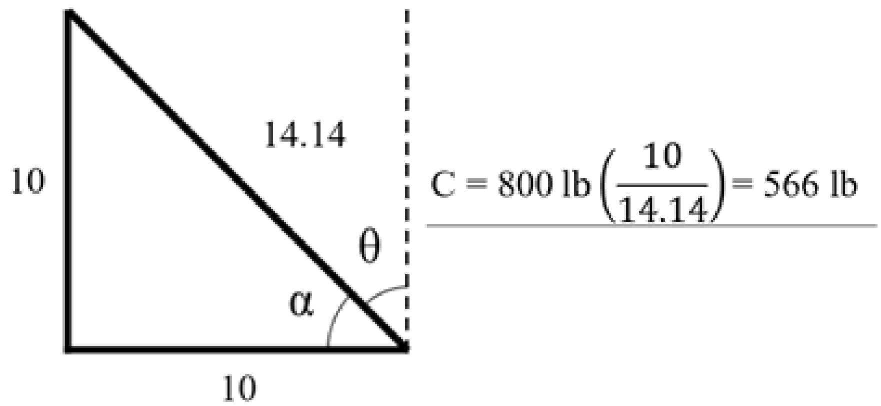


Figure D-14-3. Geometric Components of Compression Strength Value

8. **Calculate the total resisting capacity of the diagonal bracing system:**

Summarize Result for 2% Dead Load

Total resisting capacity = C + T = 566 lb + 1506 lb = 2072 lb

Resisting Capacity = 2072 lb > Horizontal Demand Force = 1900 lb

Bracing system is adequate for 2% Dead Load

Summarize Result for Wind Load

Repeat above process for wind load to calculate the Resisting Capacity, using C_D = 1.6 rather than 1.25. All other factors are the same.

The Resisting Capacity for wind load can also be derived by multiplying the resisting capacity for 2% Dead Load (above) by the ratio $\frac{C_D \text{ wind}}{C_D \text{ 2\%}} = \frac{1.6}{1.25}$

Resisting Capacity = 2072 lb $\left(\frac{1.6}{1.25}\right)$ = 2652 lb > Horizontal Demand Force = 1800 lb

Bracing system is adequate for Wind Load

Appendix D Example 15 – Diagonal Bracing of Single Tier Framed Bent – Bolted Connections

Refer to *Falsework Manual,* Section 6-3, *Diagonal Bracing* and Section 5-3, *Timber Fasteners*. This example demonstrates how to determine if the bracing system of a single tier framed bent is adequate. All connections are bolted.

Given Information

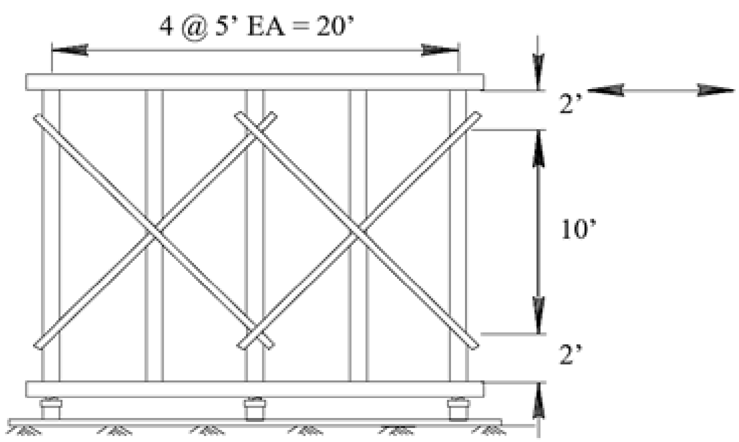


Figure D-15-1. Single Tier Framed Bent with Multiple Diagonal Bracing

2% Dead Load = 4200 lb
Wind Load = 4100 lb

Posts:
12 x 12 Rough Douglas Fir-Larch #1 (G=0.50)

Diagonal Braces:
2x8 Douglas Fir-Larch #2 (G=0.50)

Connectors:
End of brace to post: ¾ ″ Ø Bolt
Center of brace to post: ¾ ″ Ø Bolt
(All bolts in single shear)

Determine if the Bracing System is Adequate

1. **Determine the strength of the bolted connection between brace and post:**

   (See Example Problem #10 for additional information)

   Adjusted connection capacity (Z') = 1530 lb

2. **Determine strength of diagonal braces in tension:**

Reference design value in tension F_t = 575 psi (NDS supplement table 4A)

Adjustment factors from NDS table 4.3.1:

C_D = 1.25	*Duration Factor for 2% lateral loading*
C_M = 1.0	*Wet Service Factor NDS table 4A (Assume < 19% moisture content)*
C_t = 1.0	*Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)*
C_F = 1.2	*Size Factor NDS Table 4A*
C_i = 1.0	*Incising Factor NDS 4.3.8*

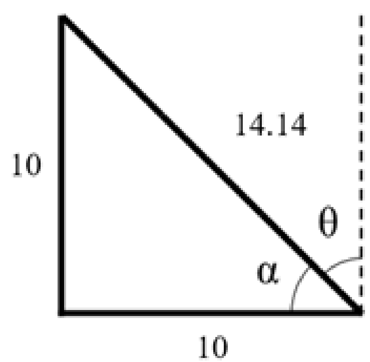
Adjusted design value Ft' = Ft (CD)(CM)(Ct)(CF)(Ci) = 863 psi

Tension capacity = 863 psi (1.5")(7.25") = 9385 lb

3. **Determine strength value of the tension members:**

9385 lb > 1530 lb ∴ Connection controls tension

4. **Calculate the horizontal component of the strength value for tension members:**



$$T = 1530 \text{ lb}\left(\frac{10}{14.14}\right) = 1082 \text{ lb}$$

Figure D-15-2. Geometric Components of Tension Strength Value

5. **Determine the capacity of diagonal brace in compression:**

Determine connection capacity of diagonal brace in compression:

Connection capacity = 1530 lb (from step 1 above.)

Determine the capacity of diagonal brace in compression:

Reference design value in compression F_c = 1350 psi (NDS supplement table 4A)

Adjustment factors from NDS table 4.3.1:

C_D = 1.25	*Duration Factor for 2% lateral loading*
C_M = 1.0	*Wet Service Factor NDS table 4A (Assume < 19% moisture content)*
C_t = 1.0	*Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)*
C_F = 1.05	*Size Factor NDS Table 4A*
C_i = 1.0	*Incising Factor NDS 4.3.8*
C_P = 0.083	*Column Stability Factor NDS 3.7.1 (unsupported length =* $\frac{14.14}{2}$ = 7.07'*)*

Adjusted design compression value F_c' = F_c $(C_D)(C_M)(C_t)(C_F)(C_i)(C_P)$ = 147 psi

Compression brace capacity = 147 psi(1.5")(7.25") = 1599 lb

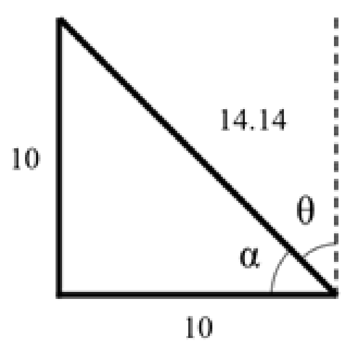
6. **Determine the strength value of the compression members**

1599 lb > 1530 lb ∴ connection controls compression

Limit to ½ theoretical strength for compression values: See section 6-3.02 *Wood Cross Bracing.*

Reduced compression brace capacity = $\frac{1530 \text{ lb}}{2}$ = 765 lb

7. **Calculate the horizontal component of the strength value for the compression member**



$$C = 765 \text{ lb}\left(\frac{10}{14.14}\right) = 541 \text{ lb}$$

Figure D-15-3. Geometric Components of Compression Strength Value

8. **Calculate the total resisting capacity of the diagonal bracing system:**

Summarize Result for 2% Dead Load

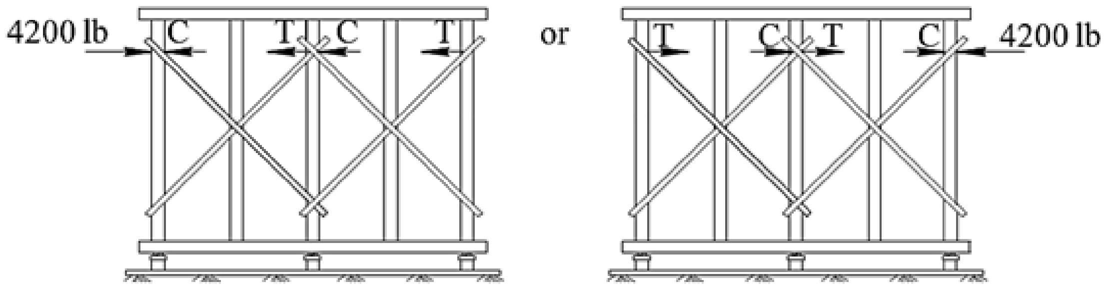


Figure D-15-4. Total Resisting Capacity for 2% Dead Load

Total resisting capacity = $\Sigma(\mathrm{C} + \mathrm{T})$ = 541 + 1082 + 541 + 1082 = 3246 lb

Resisting capacity = 3246 lb < Horizontal demand force = 4200 lb

<u>**Bracing system is inadequate for 2% Dead Load**</u>

Summarize Result for Wind Load

Repeat above process for wind load to calculate the Resisting Capacity, using C_D = 1.6 rather than 1.25. All other factors are the same.

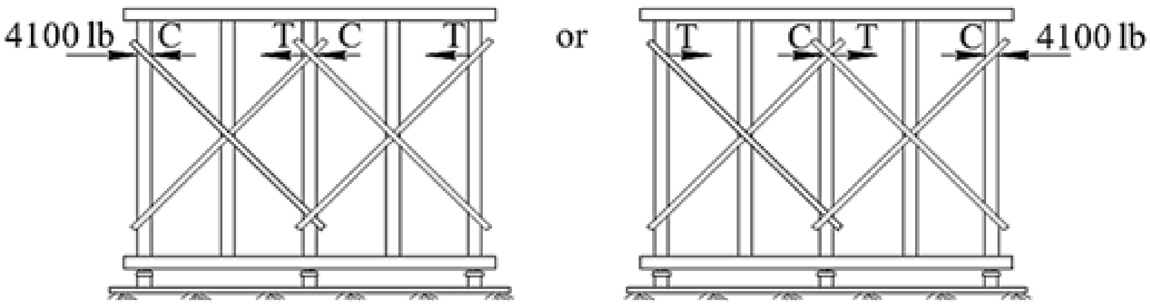


Figure D-15-5. Total Resisting Capacity for Wind Load

The Resisting Capacity for wind load can also be derived by multiplying the resisting capacity for 2% Dead Load (above table) by the ratio $\frac{C_D \text{ wind}}{C_D \text{ 2\%}} = \frac{1.6}{1.25}$

Resisting Capacity = 3246 lb $\left(\frac{1.6}{1.25}\right)$ = 4155 lb > Horizontal Demand Force = 4100 lb

<u>**Bracing system is adequate for Wind Load**</u>

Bracing system does not have enough capacity to resist <u>both</u> 2% Dead Load and Wind Load.

<u>Bracing system is inadequate.</u>

Appendix D Example 16 – Diagonal Bracing of Multi-Tiered Framed Bents – Two Posts

Refer to *Falsework Manual,* Section 6-3, *Diagonal Bracing* and Section 5-3, *Timber Fasteners*. This example demonstrates how to determine if the bracing system of a multi-tiered framed bent is adequate. The tiers are different heights. The brace to post connections are bolted, and the mid brace connections are nailed.

Given Information

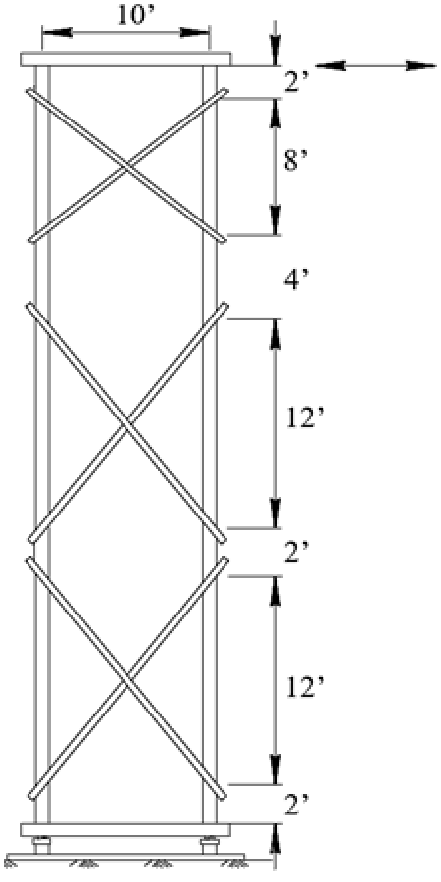


Figure D-16-1. Multi-Tiered Framed Bent

2% Dead load = 2700 lb
Wind load = 2800 lb

Posts:
12 x 12 Rough Douglas Fir-Larch #1 (G=0.50)

Diagonal Braces:
2x8 S4S Douglas Fir-Larch #2 (G=0.50)

Connectors:
Brace to Post (Top Brace): 1 x 7/8″ Ø Bolt
Brace to Post (Middle Brace): 1 x 1″ Ø Bolt
Brace to Post (Bottom Brace): 1 x 1″ Ø Bolt
Mid Brace Connections (All): 4 - 16d common nails
(All bolts in single shear)

Determine if the Bracing System is Adequate

ANALYZE THE TOP TIER

1. Determine the connection capacity between brace and post:

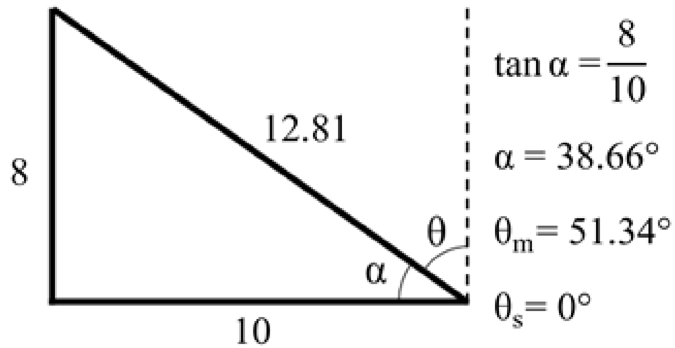


Figure D-16-2. Top Tier Member Lengths and Orientation

Main Member Properties

l_m = 12 in	*thickness (12x12)*
$t_m = l_m$ = 12 in	
θ_m= 51.34°	*angle between direction of loading & direction of grain*
G = 0.50	*Specific Gravity NDS Table 12.3.3*

Side Member Properties

l_s = 1.5 in	*thickness (2x8)*
$t_s = l_s$ = 1.5 in	
θ_s= 0°	*angle between direction of loading & direction of grain*

Connector Properties

D = 0.875 in	*connector diameter*
F_{yb} = 45000 psi	*Yield Strength (See Footnote #2 NDS table 12A)*
$F_{e.pll}$ = 11200G psi = 5600 psi	*Dowel Bearing Strength Parallel to Grain (NDS table 12.3.3 footnote 2)*
$F_{e.perp} = \dfrac{6100G^{1.45}}{\sqrt{D}}$ = 2387 psi	*Dowel Bearing Strength Perpendicular to Grain (NDS table 12.3.3 Footnote 2)*

Compare values to NDS Table 12.3.3:

$F_{e.pll\ (NDS\ Table\ 12.3.3)}$ = 5600 psi

$F_{e.perp\ (NDS\ Table\ 12.3.3)}$ = 2400 psi

Use calculated value for F_{perp} = 2387 psi

Find Dowel Bearing Strength at an Angle to Grain (NDS Section 12.3.4):

$$F_{em} = \frac{F_{e.pll}F_{perp}}{F_{e.pll}(\sin(\theta_m))^2 + F_{perp}(\cos(\theta_m))^2} = 3076 \text{ psi}$$

$$F_{es} = \frac{F_{e.pll}F_{perp}}{F_{e.pll}(\sin(\theta_s))^2 + F_{perp}(\cos(\theta_s))^2} = 5600 \text{ psi}$$

Find Reduction Term, R_d (NDS Table 12.3.1B):

$\theta = \max(\theta_m, \theta_s) = 51.34°$ *Maximum angle between direction of load and direction of grain for any member in connection (See Table 12.3.1B)*

$K_\theta = 1 + 0.25 \frac{\theta}{90 \text{ deg}} = 1.1426$

$R_{d\_I} = 4\ K_\theta = 4.57$ *Reduction Term for Yield Mode I_m and I_s*

$R_{d\_II} = 3.6\ K_\theta = 4.11$ *Reduction Term for Yield Mode II*

$R_{d\_III.IV} = 3.2\ K_\theta = 3.66$ *Reduction Term for Yield Mode III_m, III_s, and IV*

Find Yield Limit Equations for Single Shear (NDS Table 12.3.1A):

$$R_e = \frac{F_{em}}{F_{es}} = 0.5492$$

$$R_t = \frac{l_m}{l_s} = 8$$

$$k_1 = \frac{\sqrt{R_e + 2R_e^2\left(1 + R_t + R_t^2\right) + R_t^2 R_e^3} - R_e(1 + R_t)}{(1 + R_e)} = 1.6047$$

$$k_2 = -1 + \sqrt{2(1 + R_e) + \frac{2F_{yb}(1 + 2R_e)D^2}{3F_{em}l_m^2}} = 0.7909$$

$$k_3 = -1 + \sqrt{\frac{2(1 + R_e)}{R_e} + \frac{2F_{yb}(2 + R_e)D^2}{3F_{em}l_s^2}} = 2.7554$$

$$Z_{Im} = \frac{Dl_mF_{em}}{R_{d\_I}} = 7066 \text{ lb}$$ *NDS Eqn 12.3-1*

$$Z_{Is} = \frac{DI_sF_{es}}{R_{d\_I}} = 1608 \text{ lb}$$ *NDS Eqn 12.3-2*

$$Z_{II} = \frac{k_1DI_sF_{es}}{R_{d\_II}} = 2867 \text{ lb}$$ *NDS Eqn 12.3-3*

$$Z_{IIIm} = \frac{k_2DI_mF_{em}}{(1 + 2R_e)R_{d\_III.IV}} = 3329 \text{ lb}$$ *NDS Eqn 12.3-4*

$$Z_{IIIs} = \frac{k_3DI_sF_{em}}{(2 + R_e)R_{d\_III.IV}} = 1193 \text{ lb}$$ *NDS Eqn 12.3-5*

$$Z_{IV} = \frac{D^2}{R_{d\_III.IV}}\sqrt{\frac{2F_{em}F_{yb}}{3(1 + R_e)}} = 1616 \text{ lb}$$ *NDS Eqn 12.3-6*

The controlling value is the minimum single shear capacity from the above equations.

$Z_{control} = \min(Z_{Im}, Z_{Is}, Z_{II}, Z_{IIIm}, Z_{IIIs}, Z_{IV}) = 1193$ lb *(Yield Mode IIIs controls)*

Adjustment factors from NDS Table 11.3.1:

$C_D = 1.25$	*Duration Factor for 2% lateral loading*
$C_M = 1.0$	*Wet Service Factor NDS 11.3.3 (Assume < 19% moisture content)*
$C_t = 1.0$	*Temperature Factor NDS 11.3.4 (Temp up to 100ºF)*
$C_g = 1.0$	*Group Action Factor NDS 11.3.6 (Single Fastener)*
$C_\Delta = 1.0$	*Geometry Factor NDS 12.5.1 (Assume End Dist. & spacing meet Tables 12.5.1A and 12.5.1B)*
$C_{eg} = 1.0$	*End Grain Factor NDS 12.5.2 (Does not apply)*
$C_{di} = 1.0$	*Diaphragm Factor NDS 12.5.3 (Does not apply)*
$C_{tn} = 1.0$	*Toe Nail Factor NDS 12.5.4 (Does not apply)*

Adjusted lateral design value $Z' = Z(C_D)(C_M)(C_t)(C_g)(C_\Delta) = 1492$ lb

2. **Determine the capacity of the diagonal brace in tension:**

Reference design value in tension $F_t = 575$ psi (NDS supplement table 4A)

Adjustment factors from NDS table 4.3.1:

$C_D = 1.25$	*Duration Factor for 2% lateral loading*
$C_M = 1.0$	*Wet Service Factor NDS table 4A (Assume < 19% moisture content)*

C_t = 1.0 *Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)*
C_F = 1.2 *Size Factor NDS Table 4A*
C_i = 1.0 *Incising Factor NDS 4.3.8*

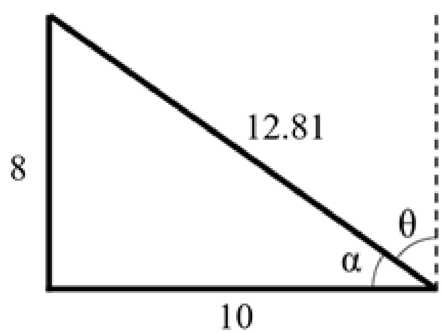
Adjusted design value $F_t' = F_t$ (CD)(CM)(Ct)(CF)(Ci) = 863 psi

Tension capacity = 863 psi(1.5")(7.25") = 9385 lb

3. Determine the strength value of the tension members:

9385 lb > 1492 lb ∴ Connection strength controls

4. Calculate the horizontal component of the strength value for the tension members:



$T = 1492 \cos \alpha = 1165$ lb

Figure D-16-3. Geometric Components of Tension Strength Value for Top Tier

5. Determine the capacity of diagonal brace in compression:

First check adequacy of the connection to reduce the unsupported length of compression member (See Section 6-3.02, *Wood Cross Bracing*):

(See Example Problem #14 Step 5 for additional information)

Connection capacity = 300 lb > 250 lb (minimum required per section 6-3.02)

Check cross brace capacity in compression:

Reference design value in compression F_c = 1350 psi (NDS supplement table 4A)

Adjustment factors from NDS table 4.3.1:

C_D = 1.25 *Duration Factor for 2% lateral loading*

C_M = 1.0 *Wet Service Factor NDS table 4A (Assume < 19% moisture content)*

C_t = 1.0 *Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)*

C_F = 1.05 *Size Factor NDS Table 4A*

C_i = 1.0 *Incising Factor NDS 4.3.8*

$$C_p = \frac{1 + (F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1 + (F_{cE}/F_c^*)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}} = 0.1003$$

Column Stability Factor NDS Eqn. 3.7-1

where:

l_e = (12.81 ft/2) = 6.405 ft = 76.86 in — *unsupported length*

d = 1.5 in — *member width, weak direction*

E_{min} = 580,000 psi — *NDS supplement table 4A*

$F_{cE} = \dfrac{0.822E_{min}'}{(l_e/d)^2} = 182$ — *NDS 3.7.1*

F_c^* = F_c $(C_D)(C_M)(C_t)(C_F)(C_i)$ = 1772 psi — *Adjusted design compression value except C_p*

c = 0.8 for sawn lumber — *NDS 3.7.1*

Adjusted design compression value $F_c' = F_c\,(C_D)(C_M)(C_t)(C_F)(C_i)(C_p)$ = 177.7 psi

Compression brace capacity = 177.7 psi (1.5")(7.25") = 1932 lb

6. **Determine the strength value of the compression members:**

Connection capacity = 1492 lb

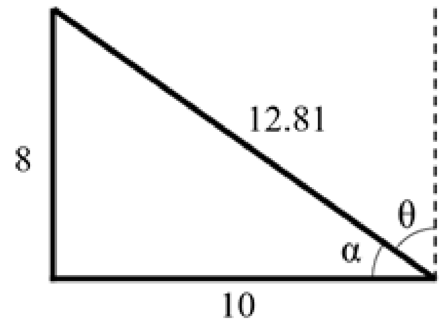
(See step 1. Capacity in tension and compression are the same)

1932 lb > 1492 lb ∴ connection controls compression

Limit to ½ theoretical strength for compression values: See Section 6-3.02, *Wood Cross Bracing.*

Reduced compression brace capacity $= \dfrac{1492 \text{ lb}}{2} = 746$ lb

7. **Calculate the horizontal component of the strength value for the compression member:**



$\underline{C = 746 \cos\alpha = 582 \text{ lb}}$

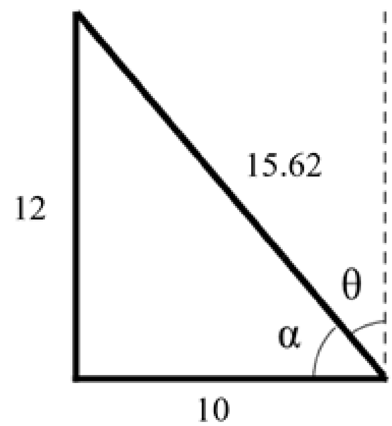
Figure D-16-4. Geometric Components of Compression Strength Value for Top Tier

8. **Calculate the total resisting capacity of the top tier of bracing:**

<u>Total resisting capacity = Σ(C+T) = 582 + 1165 = 1747 lb</u>

ANALYZE THE MIDDLE TIER

1. **Determine the connection capacity between brace and post:**



$\tan\alpha = \frac{12}{10}$

$\alpha = 50.1944°$

$\theta_m = 39.806°$

$\theta_s = 0°$

Figure D-16-5. Middle Tier Member Lengths and Orientation

<u>Main Member Properties</u>		<u>Side Member Properties</u>	
l_m = 12 in	*thickness (12x12)*	l_s = 1.5 in	*thickness (2x8)*
$t_m = l_m$ = 12 in		$t_s = l_s$ = 1.5 in	
θ_m= 39.806°	*angle between direction of loading & direction of grain*	θ_s= 0°	*angle between direction of loading & direction of grain*
G = 0.50	*Specific Gravity NDS Table 12.3.3*		

Connector Properties

$D = 1\ \text{in}$	*connector diameter*
$F_{yb} = 45000\ \text{psi}$	*Yield Strength (See Footnote #2 NDS table 12A)*
$F_{e.pll} = 11200G\ \text{psi} = 5600\ \text{psi}$	*Dowel Bearing Strength Parallel to Grain*
$F_{e.perp} = \dfrac{6100G^{1.45}}{\sqrt{\dfrac{D}{in}}} = 2233\ \text{psi}$	*Dowel Bearing Strength Perpendicular to Grain (See Footnote #2 NDS table 12A)*

Compare values to NDS Table 12.3.3:

$F_{e.pll\ (NDS\ Table\ 12.3.3)} = 5600\ \text{psi}$

$F_{e.perp\ (NDS\ Table\ 12.3.3)} = 2400\ \text{psi}$

Use calculated value for $F_{perp} = 2233$ psi

Find Dowel Bearing Strength at an Angle to Grain (NDS Section 12.3.4):

$$F_{em} = \frac{F_{e.pll}F_{perp}}{F_{e.pll}(\sin(\theta_m))^2 + F_{perp}(\cos(\theta_m))^2} = 3461\ \text{psi}$$

$$F_{es} = \frac{F_{e.pll}F_{perp}}{F_{e.pll}(\sin(\theta_s))^2 + F_{perp}(\cos(\theta_s))^2} = 5600\ \text{psi}$$

Use same methodology as top tier to find controlling yield mode and adjusted lateral design value:

$Z_{control} = \min(Z_{Im}, Z_{Is}, Z_{II}, Z_{IIIm}, Z_{IIIs}, Z_{IV}) = 1626$ lb *(Yield Mode IIIs controls)*

Adjusted lateral design value $Z' = Z(C_D)(C_M)(C_t)(C_g)(C_\Delta) = 1626\ (1.25) = 2033$ lb

2. **Determine the capacity of the diagonal brace in tension**

   By inspection, same as top tier. See top tier, step #2.

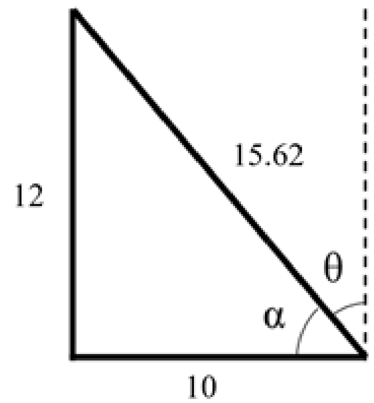
   Adjusted design value $F_t' = F_t\ (C_D)(C_M)(C_t)(C_F)(C_i) = 863$ psi

   Tension capacity = 863 psi(1.5”)(7.25”) = 9385 lb

3. **Determine the strength value of the tension members**

   9385 lb > 2033 lb ∴ Connection strength controls

4. Calculate the horizontal component of the strength value for the tension members



T = 2033 cos α = 1301 lb

Figure D-16-6. Geometric Components of Tension Strength Value for Middle Tier

5. Determine the capacity of diagonal brace in compression:

Adequacy of connection to reduce unsupported length of compression member was checked previously, see Step 5 of Top Tier.

Check cross brace capacity in compression:

Reference design value in compression F_c = 1350 psi (NDS supplement table 4A)

Adjustment factors from NDS table 4.3.1:

C_D	= 1.25	*Duration Factor for 2% lateral loading*
C_M	= 1.0	*Wet Service Factor NDS table 4A (Assume < 19% moisture content)*
C_t	= 1.0	*Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)*
C_F	= 1.05	*Size Factor NDS Table 4A*
C_i	= 1.0	*Incising Factor NDS 4.3.8*

$$C_p = \frac{1 + (F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1 + (F_{cE}/F_c^*)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}} = 0.0679$$

Column Stability Factor
NDS Eqn. 3.7-1

where:

l_e	= (15.62'/2) = 7.81 ft = 93.72 in	*unsupported length*
d	= 1.5 in	*member width, weak direction*
E_{min}	= 580,000 psi	*NDS supplement table 4A*

F_{cE}	$= \dfrac{0.822E_{min}{'}}{(l_e/d)^2} = 122$	*NDS 3.7.1*
F_c^*	$= F_c\,(C_D)(C_M)(C_t)(C_F)(C_i) = 1772$ psi	*Adjusted design compression value except C_p*
c	= 0.8 for sawn lumber	*NDS 3.7.1*

Adjusted design compression value $F_c' = F_c\,(C_D)(C_M)(C_t)(C_F)(C_i)(C_p) = 120.4$ psi

Compression brace capacity = 120.4 psi (1.5")(7.25") = 1309 lb

6. **Determine the strength value of the compression members:**

   Connection capacity = 2033 lb

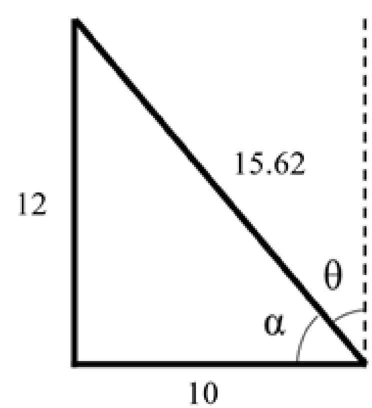
   (See step 1. Capacity in tension and compression are the same)

   1309 lb < 2033 lb ∴ member controls compression

   Limit to 1/2 theoretical strength for compression values: See section 6-3.02, *Wood Cross Bracing.*

   Reduced compression brace capacity = $\dfrac{1309\text{ lb}}{2}$ = 655 lb

7. **Calculate the horizontal component of the strength value for the compression member:**



C = 655 cos α = 419 lb

Figure D-16-7. Geometric Components of Compression Strength Value for Middle Tier

8. **Calculate the total resisting capacity of the middle tier of bracing:**

   Total resisting capacity = Σ(C+T) = 419 + 1301 = 1720 lb

ANALYZE THE BOTTOM TIER

Since the bottom tier is identical to the middle tier, the resisting capacity is equal to the middle tier. By inspection, Σ(C+T) = 1720 lb.

Summary

Summarize Results for 2% Dead Load:

Tier	Resisting Capacity	Collapsing Force = 2700 lb
Top	1747 lb	No Good
Middle	1720 lb	No Good
Bottom	1720 lb	No Good

Summarize Results for Wind Load:

Repeat above process for wind load to calculate the Resisting Capacity, using C_D = 1.6 rather than 1.25. All other factors are the same.

The Resisting Capacity for wind load can also be derived by multiplying the resisting capacity for 2% Dead Load (above table) by the factor $\frac{C_D \text{ wind}}{C_D \text{ 2\%}} = \frac{1.6}{1.25}$

Σ(C+T) Top Tier = 1747 lb $\left(\frac{1.6}{1.25}\right)$ = 2236 lb

Σ(C+T) Middle & Bottom Tiers = 1720 lb $\left(\frac{1.6}{1.25}\right)$ = 2202 lb

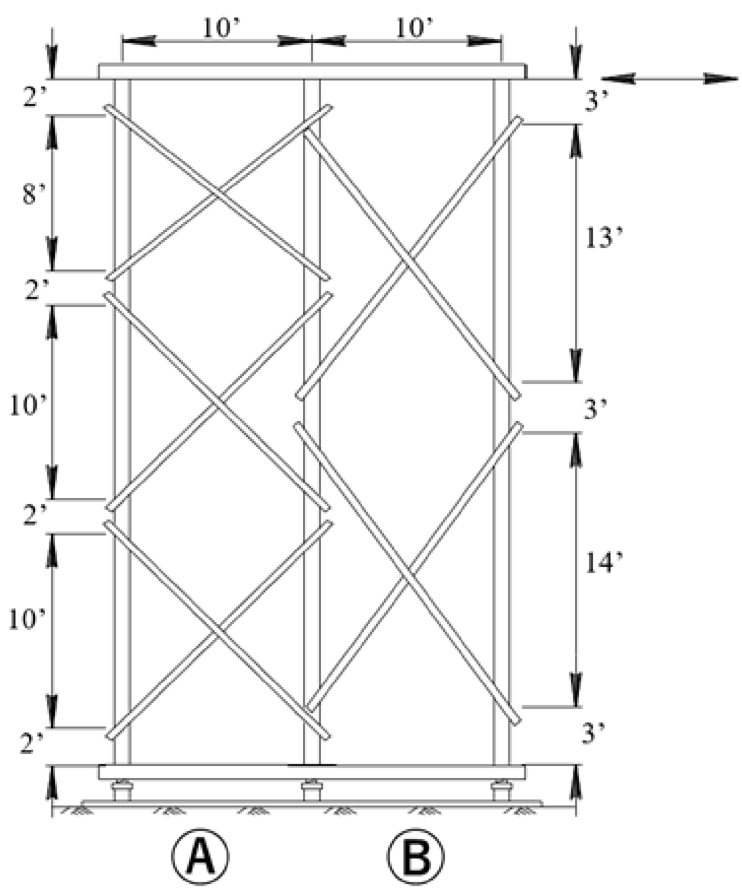
Tier	Resisting Capacity	Collapsing Force = 2800 lb
Top	2236 lb	No Good
Middle	2202 lb	No Good
Bottom	2202 lb	No Good

Bracing system is inadequate.

Appendix D Example 17 – Diagonal Bracing of Multi-Tiered Framed Bents – Multiple Posts

Refer to *Falsework Manual,* Section 6-3, *Diagonal Bracing* and Section 5-3, *Timber Fasteners*. This example demonstrates how to determine if the bracing system of a multi-tiered framed bent is adequate. The falsework bent has multiple posts, and the tiers are different heights. The brace to post connections and mid brace connections are bolted.

Given Information



2% Dead load = 3500 lb
Wind load = 3200 lb

Posts:
12 x 12 Rough Douglas Fir-Larch #2
(G=0.50)

Diagonal Braces:
2x8 S4S Douglas Fir-Larch #2
(G=0.50)

Connectors:
Brace to Post: 1x3/4″ Ø bolt
Intersection of brace: 1x3/4″ Ø bolt
(All bolts in single shear)

Figure D-17-1. Multi-Tiered Framed Bent with Multiple Posts

Determine if the Bracing System is Adequate

ANALYZE THE TOP TIER IN BRACING □

1. Determine the connection capacity between brace and post

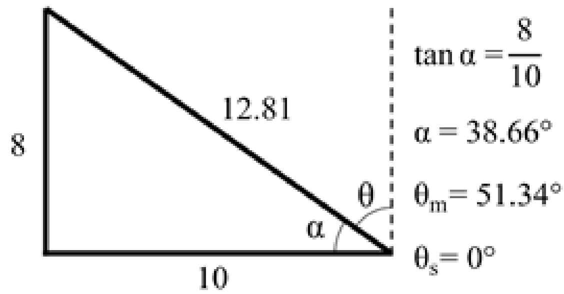


Figure D-17-2. Bracing Ⓐ Top Tier Member Lengths and Orientation

Main Member Properties

l_m = 12 in	*thickness (12x12)*
$t_m = l_m$ = 12 in	
θ_m= 51.34°	*angle between direction of loading & direction of grain*
G = 0.50	*Specific Gravity NDS Table 12.3.3*

Side Member Properties

l_s = 1.5 in	*thickness (2x8)*
$t_s = l_s$ = 1.5 in	
θ_s= 0°	*angle between direction of loading & direction of grain*

Connector Properties

D = 0.75 in	*connector diameter*
F_{yb} = 45000 psi	*Yield Strength (See Footnote #2 of Bolt Tables)*
$F_{e.pll}$ = 11200G psi = 5600 psi	*Dowel Bearing Strength Parallel to Grain*
$F_{e.perp} = \dfrac{6100G^{1.45}}{\sqrt{D}}$ = 2578 psi	*Dowel Bearing Strength Perpendicular to Grain*

Compare values to NDS Table 12.3.3:

$F_{e.pll\ (NDS\ Table\ 12.3.3)} = 5600$ psi

$F_{e.perp\ (NDS\ Table\ 12.3.3)} = 2600$ psi

Use calculated value for $F_{perp} = 2578$ psi

Find Dowel Bearing Strength at an Angle to Grain (NDS Section 12.3.4):

$$F_{em} = \frac{F_{e.pll}F_{perp}}{F_{e.pll}(\sin(\theta_m))^2 + F_{perp}(\cos(\theta_m))^2} = 3266 \text{ psi}$$

$$F_{es} = \frac{F_{e.pll}F_{perp}}{F_{e.pll}(\sin(\theta_s))^2 + F_{perp}(\cos(\theta_s))^2} = 5600 \text{ psi}$$

Find Reduction Term, R_d (NDS Table 12.3.1B):

$\theta = \max(\theta_m, \theta_s) = 51.34°$ *Maximum angle between direction of load and direction of grain for any member in connection (See Table 12.3.1B)*

$$K_\theta = 1+0.25\frac{\theta}{90 \text{ deg}} = 1.1426$$

$R_{d\_I} = 4\ K_\theta = 4.57$ *Reduction Term for Yield Mode I_m and I_s*

$R_{d\_II} = 3.6\ K_\theta = 4.11$ *Reduction Term for Yield Mode II*

$R_{d\_III.IV} = 3.2\ K_\theta = 3.66$ *Reduction Term for Yield Mode III_m, III_s, and IV*

Find Yield Limit Equations for Single Shear (NDS Table 12.3.1A):

$$R_e = \frac{F_{em}}{F_{es}} = 0.5832$$

$$R_t = \frac{l_m}{l_s} = 8$$

$$k_1 = \frac{\sqrt{R_e + 2R_e^2\left(1 + R_t + R_t^2\right) + R_t^2R_e^3} - R_e(1 + R_t)}{(1 + R_e)} = 1.6956$$

$$k_2 = -1 + \sqrt{2(1 + R_e) + \frac{2F_{yb}(1 + 2R_e)D^2}{3F_{em}l_m^2}} = 0.8011$$

$$k_3 = -1 + \sqrt{\frac{2(1 + R_e)}{R_e} + \frac{2F_{yb}(2 + R_e)D^2}{3F_{em}l_s^2}} = 2.3707$$

$$Z_{Im} = \frac{Dl_mF_{em}}{R_{d\_I}} = 6431 \text{ lb}$$ *NDS Eqn 12.3-1*

$$Z_{Is} = \frac{Dl_sF_{es}}{R_{d\_I}} = 1378 \text{ lb}$$ *NDS Eqn 12.3-2*

$$Z_{II} = \frac{k_1Dl_sF_{es}}{R_{d\_II}} = 2597 \text{ lb}$$ *NDS Eqn 12.3-3*

$$Z_{IIIm} = \frac{k_2Dl_mF_{em}}{(1 + 2R_e)R_{d\_III.IV}} = 2973 \text{ lb}$$ *NDS Eqn 12.3-4*

$$Z_{IIIs} = \frac{k_3Dl_sF_{em}}{(2 + R_e)R_{d\_III.IV}} = 922 \text{ lb}$$ *NDS Eqn 12.3-5*

$$Z_{IV} = \frac{D^2}{R_{d\_III.IV}}\sqrt{\frac{2F_{em}F_{yb}}{3(1 + R_e)}} = 1210 \text{ lb}$$ *NDS Eqn 12.3-6*

The controlling value is the minimum single shear capacity from the above equations.

$Z_{control}$ = min (Z_{Im}, Z_{Is}, Z_{II}, Z_{IIIm}, Z_{IIIs}, Z_{IV}) = 922 lb *(Yield Mode IIIs controls)*

Adjustment factors from NDS Table 11.3.1:

C_D = 1.25 *Duration Factor for 2% lateral loading*
C_M = 1.0 *Wet Service Factor NDS 11.3.3 (Assume < 19% moisture content)*
C_t = 1.0 *Temperature Factor NDS 11.3.4 (Temp up to 100ºF)*
C_g = 1.0 *Group Action Factor NDS 11.3.6 (Single Fastener)*
C_Δ = 1.0 *Geometry Factor NDS 12.5.1 (Assume End Dist. & spacing meet Tables 12.5.1A and 12.5.1B)*
C_{eg} = 1.0 *End Grain Factor NDS 12.5.2 (Does not apply)*
C_{di} = 1.0 *Diaphragm Factor NDS 12.5.3 (Does not apply)*
C_{tn} = 1.0 *Toe Nail Factor NDS 12.5.4 (Does not apply)*

Adjusted lateral design value Z' = Z(C_D)(C_M)(C_t)(C_g)(C_Δ) = 1153 lb

2. Determine the capacity of the diagonal brace in tension

Reference design value in tension F_t = 575 psi (NDS supplement table 4A)

Adjustment factors from NDS table 4.3.1:

C_D = 1.25	*Duration Factor for 2% lateral loading*
C_M = 1.0	*Wet Service Factor NDS table 4A (Assume < 19% moisture content)*
C_t = 1.0	*Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)*
C_F = 1.2	*Size Factor NDS Table 4A*
C_i = 1.0	*Incising Factor NDS 4.3.8*

Adjusted design value $F_t' = F_t\,(C_D)(C_M)(C_t)(C_F)(C_i)$ = 862.5 psi

Tension capacity = 862.5 psi(1.5")(7.25") = 9380 lb

3. **Determine the strength value of the tension members**

9380 lb > 1153 lb ∴ Connection strength controls

4. **Calculate the horizontal component of the strength value for the tension members**

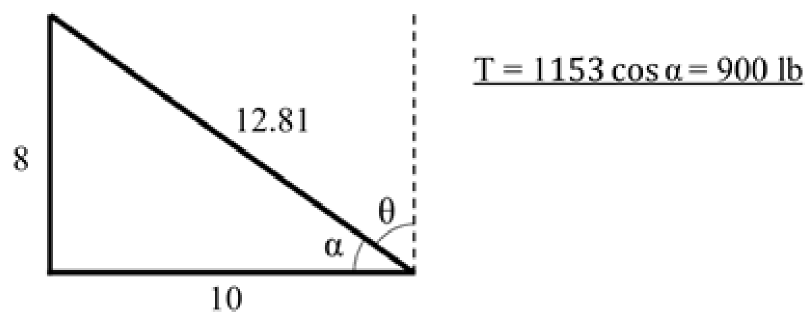


Figure D-17-3. Geometric Components of Tension Strength Value for Bracing Ⓐ Top Tier

5. **Determine the capacity of diagonal brace in compression**

Check cross brace capacity in compression:

Reference design value in compression F_c = 1350 psi (NDS supplement table 4A)

Adjustment factors from NDS table 4.3.1:

C_D = 1.25	*Duration Factor for 2% lateral loading*
C_M = 1.0	*Wet Service Factor NDS table 4A (Assume < 19% moisture content)*
C_t = 1.0	*Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)*

C_F = 1.05 *Size Factor NDS Table 4A*

C_i = 1.0 *Incising Factor NDS 4.3.8*

$$C_p = \frac{1 + (F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1 + (F_{cE}/F_c^*)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}} = 0.1003$$

Column Stability Factor NDS Eqn. 3.7-1

where:

l_e	= (12.81'/2) = 6.405' = 76.86"	*unsupported length*
d	= 1.5"	*member width, weak direction*
E_{min}	= 580,000 psi	*NDS supplement table 4A*
F_{cE}	$= \dfrac{0.822E_{min}'}{(l_e/d)^2} = 182$	*NDS 3.7.1*
F_c^*	= $F_c\,(C_D)(C_M)(C_t)(C_F)(C_i)$ = 1772 psi	*Adjusted design compression value except C_p*
c	= 0.8 for sawn lumber	*NDS 3.7.1*

Adjusted design compression value $F_c' = F_c\,(C_D)(C_M)(C_t)(C_F)(C_i)(C_p)$ = 177.7 psi

Compression brace capacity = 177.7 psi (1.5")(7.25") = 1932 lb

6. Determine the strength value of the compression members

Connection capacity = 1153 lb

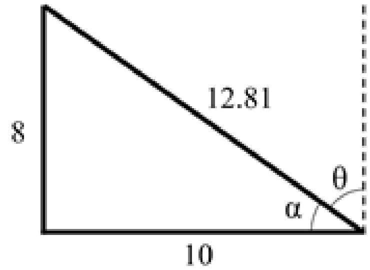
(See step 1. Capacity in tension and compression are the same)

1932 lb > 1153 lb ∴ connection controls compression

Limit to 1/2 theoretical strength for compression values: See section 6-3.02, *Wood Cross Bracing.*

Reduced compression brace capacity = $\dfrac{1153\text{ lb}}{2}$ = 576 lb

7. Calculate the horizontal component of the strength value for the compression member



$\underline{C = 576 \cos\alpha = 450 \text{ lb}}$

Figure D-17-4. Geometric Components of Compression Strength Value for Bracing Ⓐ Top Tier

8. **Calculate the total resisting capacity of the top tier of bracing**

   <u>Total resisting capacity = Σ(C+T) = 450 + 900 = 1350 lb</u>

ANALYZE THE MIDDLE TIER IN BRACING Ⓐ

1. **Determine the connection capacity between brace and post:**

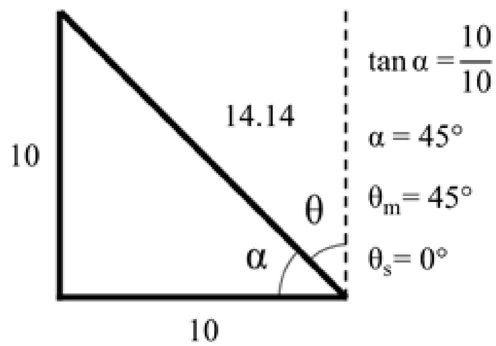


Figure D-17-5. Bracing Ⓐ Middle Tier Member Lengths and Orientation

<u>Main Member Properties</u>

l_m = 12 in	*thickness (12x12)*
$t_m = l_m$ = 12 in	
θ_m= 45°	*angle between direction of loading & direction of grain*
G = 0.50	*Specific Gravity*
	NDS Table 12.3.3

<u>Side Member Properties</u>

l_s = 1.5 in	*thickness (2x8)*
$t_s = l_s$ = 1.5 in	
θ_s= 0°	*angle between direction of loading & direction of grain*

Connector Properties

By inspection, same properties as previous tier. $F_{e.pll} = 5600$ psi $F_{e.perp} = 2578$ psi

Find Dowel Bearing Strength at an Angle to Grain (NDS Section 12.3.4):

$$F_{em} = \frac{F_{e.pll}F_{perp}}{F_{e.pll}(\sin(\theta_m))^2 + F_{perp}(\cos(\theta_m))^2} = 3531 \text{ psi}$$

$$F_{es} = \frac{F_{e.pll}F_{perp}}{F_{e.pll}(\sin(\theta_s))^2 + F_{perp}(\cos(\theta_s))^2} = 5600 \text{ psi}$$

Find Reduction Term, R_d (NDS Table 12.3.1B):

$\theta = \max(\theta_m, \theta_s) = 45°$ *Maximum angle between direction of load and direction of grain for any member in connection (See Table 12.3.1B)*

$K_\theta = 1 + 0.25 \frac{\theta}{90 \text{ deg}} = 1.125$

$R_{d\_I} = 4\ K_\theta = 4.5$ *Reduction Term for Yield Mode I_m and I_s*

$R_{d\_II} = 3.6\ K_\theta = 4.05$ *Reduction Term for Yield Mode II*

$R_{d\_III.IV} = 3.2\ K_\theta = 3.6$ *Reduction Term for Yield Mode III_m, III_s, and IV*

Find Yield Limit Equations for Single Shear (NDS Table 12.3.1A):

$$R_e = \frac{F_{em}}{F_{es}} = 0.6305$$

$$R_t = \frac{l_m}{l_s} = 8$$

$$k_1 = \frac{\sqrt{R_e + 2R_e^2\left(1 + R_t + R_t^2\right) + R_t^2 R_e^3} - R_e(1 + R_t)}{(1 + R_e)} = 1.8209$$

$$k_2 = -1 + \sqrt{2(1 + R_e) + \frac{2F_{yb}(1 + 2R_e)D^2}{3F_{em}l_m^2}} = 0.8265$$

$$k_3 = -1 + \sqrt{\frac{2(1 + R_e)}{R_e} + \frac{2F_{yb}(2 + R_e)D^2}{3F_{em}l_s^2}} = 2.2802$$

$$Z_{Im} = \frac{Dl_mF_{em}}{R_{d\_I}} = 7062 \text{ lb}$$ *NDS Eqn 12.3-1*

$$Z_{Is} = \frac{Dl_sF_{es}}{R_{d\_I}} = 1400 \text{ lb}$$ *NDS Eqn 12.3-2*

$$Z_{II} = \frac{k_1Dl_sF_{es}}{R_{d\_II}} = 2833 \text{ lb}$$ *NDS Eqn 12.3-3*

$$Z_{IIIm} = \frac{k_2Dl_mF_{em}}{(1 + 2R_e)R_{d\_III.IV}} = 3227 \text{ lb}$$ *NDS Eqn 12.3-4*

$$Z_{IIIs} = \frac{k_3Dl_sF_{em}}{(2 + R_e)R_{d\_III.IV}} = 956 \text{ lb}$$ *NDS Eqn 12.3-5*

$$Z_{IV} = \frac{D^2}{R_{d\_III.IV}}\sqrt{\frac{2F_{em}F_{yb}}{3(1 + R_e)}} = 1259 \text{ lb}$$ *NDS Eqn 12.3-6*

The controlling value is the minimum single shear capacity from the above equations.

$Z_{control}$ = min (Z_{Im}, Z_{Is}, Z_{II}, Z_{IIIm}, Z_{IIIs}, Z_{IV}) = 956 lb *(Yield Mode IIIs controls)*

Adjustment factors from NDS Table 11.3.1:

C_D = 1.25 *Duration Factor for 2% lateral loading*

C_M = 1.0 *Wet Service Factor NDS 11.3.3 (Assume < 19% moisture content)*

C_t = 1.0 *Temperature Factor NDS 11.3.4 (Temp up to 100ºF)*

C_g = 1.0 *Group Action Factor NDS 11.3.6 (Single Fastener)*

C_Δ = 1.0 *Geometry Factor NDS 12.5.1 (Assume End Dist. & spacing meet Tables 12.5.1A and 12.5.1B)*

C_{eg} = 1.0 *End Grain Factor NDS 12.5.2 (Does not apply)*

C_{di} = 1.0 *Diaphragm Factor NDS 12.5.3 (Does not apply)*

C_{tn} = 1.0 *Toe Nail Factor NDS 12.5.4 (Does not apply)*

Adjusted lateral design value Z' = Z(C_D)(C_M)(C_t)(C_g)(C_Δ) = 1196 lb

2. Determine the capacity of the diagonal brace in tension

By inspection, same as previous tier. See top tier, step #2.

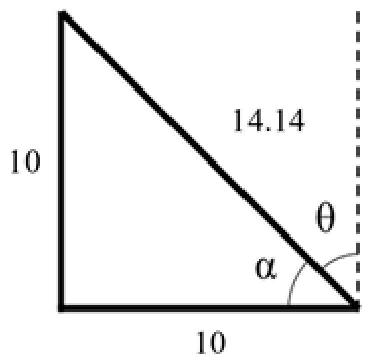
Adjusted design value $F_t' = F_t (C_D)(C_M)(Ct)(C_F)(C_i)$ = 862.5 psi

Tension capacity = 862.5 psi(1.5”)(7.25”) = 9380 lb

3. Determine the strength value of the tension members

9380 lb > 1196 lb ∴ Connection strength controls

4. Calculate the horizontal component of the strength value for the tension members



$T = 1196 \cos\alpha = 845$ lb

Figure D-17-6. Geometric Components of Tension Strength Value for Bracing Ⓐ Middle Tier

5. Determine the capacity of diagonal brace in compression

Check cross brace capacity in compression:

Reference design value in compression F_c = 1350 psi (NDS supplement table 4A)

Adjustment factors from NDS table 4.3.1:

C_D = 1.25 *Duration Factor for 2% lateral loading*
C_M = 1.0 *Wet Service Factor NDS table 4A (Assume < 19% moisture content)*
C_t = 1.0 *Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)*
C_F = 1.05 *Size Factor NDS Table 4A*
C_i = 1.0 *Incising Factor NDS 4.3.8*

$$C_p = \frac{1 + (F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1 + (F_{cE}/F_c^*)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}} = 0.0826$$

Column Stability Factor NDS Eqn. 3.7-1 where:

l_e	= (14.14'/2) = 7.071' = 84.85"	*unsupported length*
d	= 1.5"	*member width, weak direction*
E_{min}	= 580,000 psi	*NDS supplement table 4A*
F_{cE}	$= \dfrac{0.822E_{min}'}{(l_e/d)^2} = 149$	*NDS 3.7.1*
F_c^*	$= F_c\,(C_D)(C_M)(C_t)(C_F)(C_i) = 1772$ psi	*Adjusted design compression value except C_p*
c	= 0.8 for sawn lumber	*NDS 3.7.1*

Adjusted design compression value $F_c' = F_c\,(C_D)(C_M)(C_t)(C_F)(C_i)(C_p) = 146.4$ psi

Compression brace capacity = 146.4 psi (1.5")(7.25") = 1592 lb

6. **Determine the strength value of the compression members**

Connection capacity = 1196 lb

(See step 1. Capacity in tension and compression are the same)

1196 lb < 1592 lb ∴ connection controls compression

Limit to ½ theoretical strength for compression values: See section 6-3.02, *Wood Cross Bracing.*

Reduced compression brace capacity = $\dfrac{1196 \text{ lb}}{2}$ = 598 lb

7. **Calculate the horizontal component of the strength value for the compression member**

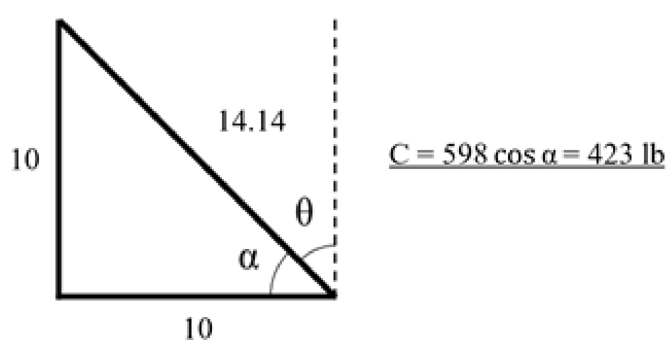


Figure D-17-7. Geometric Components of Compression Strength Value for Bracing Ⓐ Middle Tier

8. **Calculate the total resisting capacity of the top tier of bracing**

   <u>Total resisting capacity = Σ(C+T) = 423 + 845 = 1268 lb</u>

ANALYZE THE BOTTOM TIER IN BRACING A

By inspection, middle tier and bottom tier are the same.

Total resisting capacity = 1268 lb

ANALYZE THE TOP TIER IN BRACING B

1. **Determine the connection capacity between brace and post**

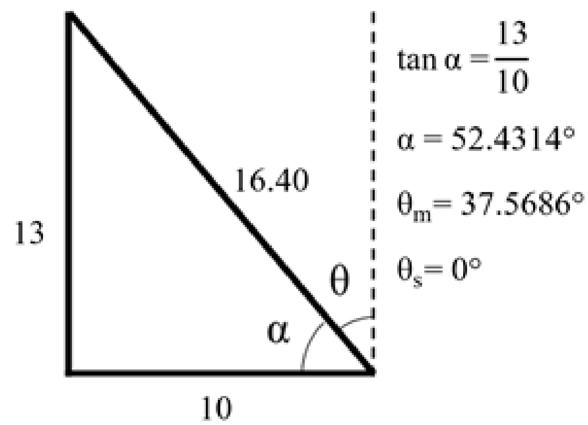


Figure D-17-8. Bracing B Top Tier Member Lengths and Orientation

<u>Main Member Properties</u>

l_m = 12 in	*thickness (12x12)*
$t_m = l_m$ = 12 in	
θ_m= 37.56°	*angle between direction of loading & direction of grain*
G = 0.50	*Specific Gravity NDS Table 12.3.3*

<u>Side Member Properties</u>

l_s = 1.5 in	*thickness (2x8)*
$t_s = l_s$ = 1.5 in	
θ_s= 0°	*angle between direction of loading & direction of grain*

<u>Connector Properties</u>

By inspection, same properties as previous tiers. $F_{e.pll}$ = 5600 psi $F_{e.perp}$ = 2578 psi

<u>Find Dowel Bearing Strength at an Angle to Grain (NDS Section 12.3.4)</u>

$$F_{em} = \frac{F_{e.pll}F_{perp}}{F_{e.pll}(\sin(\theta_m))^2 + F_{perp}(\cos(\theta_m))^2} = 3900\ \text{psi}$$

$$F_{es} = \frac{F_{e.pll}F_{perp}}{F_{e.pll}(\sin(\theta_s))^2 + F_{perp}(\cos(\theta_s))^2} = 5600\ \text{psi}$$

Find Reduction Term, R_d (NDS Table 12.3.1B):

$\theta = \max(\theta_m, \theta_s) = 37.5686°$ *Maximum angle between direction of load and direction of grain for any member in connection (See Table 12.3.1B)*

$$K_\theta = 1 + 0.25\frac{\theta}{90\ \text{deg}} = 1.1044$$

$R_{d\_I} = 4\ K_\theta = 4.42$ *Reduction Term for Yield Mode I_m and I_s*

$R_{d\_II} = 3.6\ K_\theta = 3.98$ *Reduction Term for Yield Mode II*

$R_{d\_III.IV} = 3.2\ K_\theta = 3.53$ *Reduction Term for Yield Mode III_m, III_s, and IV*

Find Yield Limit Equations for Single Shear (NDS Table 12.3.1A):

$$R_e = \frac{F_{em}}{F_{es}} = 0.6965$$

$$R_t = \frac{l_m}{l_s} = 8$$

$$k_1 = \frac{\sqrt{R_e + 2R_e^2\left(1 + R_t + R_t^2\right) + R_t^2R_e^3} - R_e(1 + R_t)}{(1 + R_e)} = 1.9940$$

$$k_2 = -1 + \sqrt{2(1 + R_e) + \frac{2F_{yb}(1 + 2R_e)D^2}{3F_{em}l_m^2}} = 0.8614$$

$$k_3 = -1 + \sqrt{\frac{2(1 + R_e)}{R_e} + \frac{2F_{yb}(2 + R_e)D^2}{3F_{em}l_s^2}} = 2.1712$$

$$Z_{Im} = \frac{Dl_mF_{em}}{R_{d\_I}} = 7947\ \text{lb}$$ *NDS Eqn 12.3-1*

$Z_{Is} = \dfrac{DI_sF_{es}}{R_{d\_I}} = 1426 \text{ lb}$ *NDS Eqn 12.3-2*

$Z_{II} = \dfrac{k_1DI_sF_{es}}{R_{d\_II}} = 3160 \text{ lb}$ *NDS Eqn 12.3-3*

$Z_{IIIm} = \dfrac{k_2DI_mF_{em}}{(1 + 2R_e)R_{d\_III.IV}} = 3576 \text{ lb}$ *NDS Eqn 12.3-4*

$Z_{IIIs} = \dfrac{k_3DI_sF_{em}}{(2 + R_e)R_{d\_III.IV}} = 1000 \text{ lb}$ *NDS Eqn 12.3-5*

$Z_{IV} = \dfrac{D^2}{R_{d\_III.IV}}\sqrt{\dfrac{2F_{em}F_{yb}}{3(1 + R_e)}} = 1322 \text{ lb}$ *NDS Eqn 12.3-6*

The controlling value is the minimum single shear capacity from the above equations.

$Z_{control} = \min(Z_{Im}, Z_{Is}, Z_{II}, Z_{IIIm}, Z_{IIIs}, Z_{IV}) = 1000 \text{ lb}$ *(Yield Mode IIIs controls)*

Adjustment factors from NDS Table 11.3.1:

$C_D = 1.25$	*Duration Factor for 2% lateral loading*
$C_M = 1.0$	*Wet Service Factor NDS 11.3.3 (Assume < 19% moisture content)*
$C_t = 1.0$	*Temperature Factor NDS 11.3.4 (Temp up to 100ºF)*
$C_g = 1.0$	*Group Action Factor NDS 11.3.6 (Single Fastener)*
$C_\Delta = 1.0$	*Geometry Factor NDS 12.5.1 (Assume End Dist. & spacing meet Tables 12.5.1A and 12.5.1B)*
$C_{eg} = 1.0$	*End Grain Factor NDS 12.5.2 (Does not apply)*
$C_{di} = 1.0$	*Diaphragm Factor NDS 12.5.3 (Does not apply)*
$C_{tn} = 1.0$	*Toe Nail Factor NDS 12.5.4 (Does not apply)*

Adjusted lateral design value $Z' = Z(C_D)(C_M)(C_t)(C_g)(C_\Delta) = 1250 \text{ lb}$

2. Determine the capacity of the diagonal brace in tension

By inspection, same as previous tiers. See top tier, step #2.

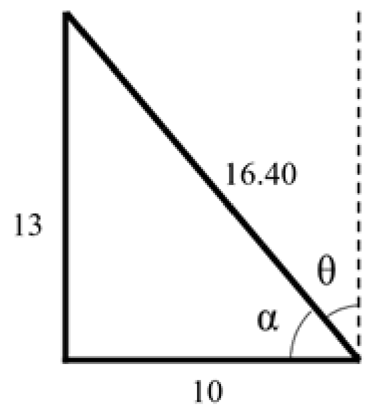
Adjusted design value Ft’ = Ft (CD)(CM)(Ct)(CF)(Ci) = 862.5 psi

Tension capacity = 862.5 psi(1.5”)(7.25”) = 9380 lb

3. Determine the strength value of the tension members

9380 lb > 1250 lb ∴ Connection strength controls

4. Calculate the horizontal component of the strength value for the tension members



T = 1250 cos α = 762 lb

Figure D-17-9. Geometric Components of Tension Strength Value for Bracing Ⓑ Top Tier

5. Determine the capacity of diagonal brace in compression

Check cross brace capacity in compression:

Reference design value in compression F_c = 1350 psi (NDS supplement table 4A)

Adjustment factors from NDS table 4.3.1:

C_D	= 1.25	*Duration Factor for 2% lateral loading*
C_M	= 1.0	*Wet Service Factor NDS table 4A (Assume < 19% moisture content)*
C_t	= 1.0	*Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)*
C_F	= 1.05	*Size Factor NDS Table 4A*
C_i	= 1.0	*Incising Factor NDS 4.3.8*

$$C_p = \frac{1 + (F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1 + (F_{cE}/F_c^*)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}} = 0.0617$$

Column Stability Factor NDS Eqn. 3.7-1

where:

l_e = (16.40'/2) = 8.20’ = 98.40” *unsupported length*

d	= 1.5”	*member width, weak direction*
E_{min}	= 580,000 psi	*NDS supplement table 4A*
F_{cE}	$= \dfrac{0.822E_{min}'}{(l_e/d)^2} = 111$	*NDS 3.7.1*
F_c^*	$= F_c\,(C_D)(C_M)(C_t)(C_F)(C_i) = 1772$ psi	*Adjusted design compression value except C_p*
c	= 0.8 for sawn lumber	*NDS 3.7.1*

Adjusted design compression value $F_c' = F_c\,(C_D)(C_M)(C_t)(C_F)(C_i)(C_p) = 109.3$ psi

Compression brace capacity = 109.3 psi (1.5”)(7.25”) = 1189 lb

6. Determine the strength value of the compression members

Connection capacity = 1250 lb

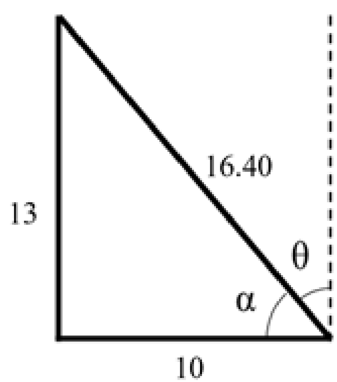
(See step 1. Capacity in tension and compression are the same)

1250 lb > 1189 lb ∴ member controls compression

Limit to 1/2 theoretical strength for compression values: See section 6-3.02, *Wood Cross Bracing.*

Reduced compression brace capacity $= \dfrac{1189 \text{ lb}}{2} = 595$ lb

7. Calculate the horizontal component of the strength value for the compression member



C = 595 cos α = 362.5 lb

Figure D-17-10. Geometric Components of Compression Strength Value for Bracing Ⓑ Top Tier

8. Calculate the total resisting capacity of the top tier of bracing

Total resisting capacity = Σ(C+T) = 362.5 + 762 = 1224.5 lb

ANALYZE THE BOTTOM TIER IN BRACING Ⓑ

1. Determine the connection capacity between brace and post

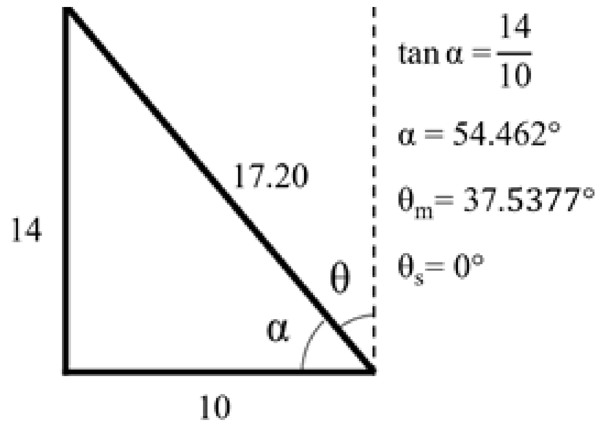


Figure D-17-11. Bracing Ⓑ Bottom Tier Member Lengths and Orientation

Main Member Properties

l_m = 12 in	*thickness (12 x 12)*
$t_m = l_m$ = 12 in	
θ_m= 35.54°	*angle between direction of loading & direction of grain*
G = 0.50	*Specific Gravity NDS Table 12.3.3*

Side Member Properties

l_s = 1.5 in	*thickness (2 x 8)*
$t_s = l_s$ = 1.5 in	
θ_s= 0°	*angle between direction of loading & direction of grain*

Connector Properties

By inspection, same properties as previous tiers. $F_{e.pll}$ = 5600 psi $F_{e.perp}$ = 2578 psi

Find Dowel Bearing Strength at an Angle to Grain (NDS Section 12.3.4):

$$F_{em} = \frac{F_{e.pll}F_{perp}}{F_{e.pll}(\sin(\theta_m))^2 + F_{perp}(\cos(\theta_m))^2} = 4012 \text{ psi}$$

$$F_{es} = \frac{F_{e.pll}F_{perp}}{F_{e.pll}(\sin(\theta_s))^2 + F_{perp}(\cos(\theta_s))^2} = 5600 \text{ psi}$$

Find Reduction Term, R_d (NDS Table 12.3.1B):

$\theta = \max(\theta_m, \theta_s) = 35.5377°$ *Maximum angle between direction of load and direction of grain for any member in connection (See Table 12.3.1B)*

$K_\theta = 1 + 0.25\frac{\theta}{90\ deg} = 1.0987$

$R_{d\_I} = 4\ K_\theta = 4.39$ *Reduction Term for Yield Mode I_m and I_s*

$R_{d\_II} = 3.6\ K_\theta = 3.96$ *Reduction Term for Yield Mode II*

$R_{d\_III.IV} = 3.2\ K_\theta = 3.52$ *Reduction Term for Yield Mode III_m, III_s, and IV*

Find Yield Limit Equations for Single Shear (NDS Table 12.3.1A):

$$R_e = \frac{F_{em}}{F_{es}} = 0.7163$$

$$R_t = \frac{l_m}{l_s} = 8$$

$$k_1 = \frac{\sqrt{R_e + 2R_e^2\left(1 + R_t + R_t^2\right) + R_t^2 R_e^3} - R_e(1 + R_t)}{(1 + R_e)} = 2.0456$$

$$k_2 = -1 + \sqrt{2(1 + R_e) + \frac{2F_{yb}(1 + 2R_e)D^2}{3F_{em}l_m^2}} = 0.8718$$

$$k_3 = -1 + \sqrt{\frac{2(1 + R_e)}{R_e} + \frac{2F_{yb}(2 + R_e)D^2}{3F_{em}l_s^2}} = 2.1417$$

$Z_{Im} = \frac{Dl_mF_{em}}{R_{d\_I}} = 8215\ lb$ *NDS Eqn 12.3-1*

$Z_{Is} = \frac{Dl_sF_{es}}{R_{d\_I}} = 1433\ lb$ *NDS Eqn 12.3-2*

$Z_{II} = \frac{k_1Dl_sF_{es}}{R_{d\_II}} = 3258\ lb$ *NDS Eqn 12.3-3*

$Z_{IIIm} = \frac{k_2Dl_mF_{em}}{(1 + 2R_e)R_{d\_III.IV}} = 3680\ lb$ *NDS Eqn 12.3-4*

$$Z_{IIIs} = \frac{k_3 D l_s F_{em}}{(2 + R_e) R_{d\_III.IV}} = 1012 \text{ lb}$$ *NDS Eqn 12.3-5*

$$Z_{IV} = \frac{D^2}{R_{d\_III.IV}} \sqrt{\frac{2F_{em}F_{yb}}{3(1 + R_e)}} = 1340 \text{ lb}$$ *NDS Eqn 12.3-6*

The controlling value is the minimum single shear capacity from the above equations.

$Z_{control}$ = min (Z_{Im}, Z_{Is}, Z_{II}, Z_{IIIm}, Z_{IIIs}, Z_{IV}) = 1012 lb *(Yield Mode IIIs controls)*

Adjustment factors from NDS Table 11.3.1:

C_D = 1.25	*Duration Factor for 2% lateral loading*
C_M = 1.0	*Wet Service Factor NDS 11.3.3 (Assume < 19% moisture content)*
C_t = 1.0	*Temperature Factor NDS 11.3.4 (Temp up to 100ºF)*
C_g = 1.0	*Group Action Factor NDS 11.3.6 (Single Fastener)*
C_Δ = 1.0	*Geometry Factor NDS 12.5.1 (Assume End Dist. & spacing meet Tables 12.5.1A and 12.5.1B)*
C_{eg} = 1.0	*End Grain Factor NDS 12.5.2 (Does not apply)*
C_{di} = 1.0	*Diaphragm Factor NDS 12.5.3 (Does not apply)*
C_{tn} = 1.0	*Toe Nail Factor NDS 12.5.4 (Does not apply)*

Adjusted lateral design value Z’ = Z(C_D)(C_M)(C_t)(C_g)(C_Δ) = 1265 lb

2. Determine the capacity of the diagonal brace in tension

By inspection, same as previous tiers. See top tier, step #2.

Adjusted design value F_t’ = F_t (C_D)(C_M)(C_t)(C_F)(C_i) = 862.5 psi

Tension capacity = 862.5 psi(1.5”)(7.25”) = 9380 lb

3. Determine the strength value of the tension members

9380 lb > 1265 lb ∴ Connection strength controls

4. Calculate the horizontal component of the strength value for the tension members

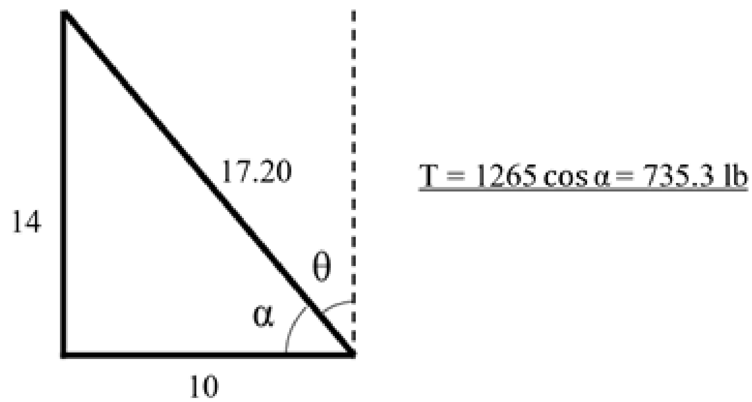


Figure D-17-12. Geometric Components of Tension Strength Value for Bracing Ⓑ Bottom Tier

5. Determine the capacity of diagonal brace in compression

Check cross brace capacity in compression:

Reference design value in compression F_c = 1350 psi (NDS supplement table 4A)

Adjustment factors from NDS table 4.3.1:

C_D	= 1.25	*Duration Factor for 2% lateral loading*
C_M	= 1.0	*Wet Service Factor NDS table 4A (Assume < 19% moisture content)*
C_t	= 1.0	*Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)*
C_F	= 1.05	*Size Factor NDS Table 4A*
C_i	= 1.0	*Incising Factor NDS 4.3.8*

$$C_p = \frac{1 + (F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1 + (F_{cE}/F_c^*)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}}$$

Column Stability Factor NDS Eqn. 3.7-1

$= 0.0561$

where:

l_e	= (17.20'/2) = 8.60’ = 103.20”	*unsupported length*
d	= 1.5”	*member width, weak direction*
E_{min}	= 580,000 psi	*NDS supplement table 4A*

F_{cE}	$= \dfrac{0.822E_{min}{}'}{(l_e/d)^2} = 101$	*NDS 3.7.1*
F_c^*	$= F_c\,(C_D)(C_M)(C_t)(C_F)(C_i) = 1772$ psi	*Adjusted design compression value except C_p*
c	= 0.8 for sawn lumber	*NDS 3.7.1*

Adjusted design compression value $F_c' = F_c\,(C_D)(C_M)(C_t)(C_F)(C_i)(C_p) = 99.5$ psi

Compression brace capacity = 99.5 psi (1.5”)(7.25”) = 1081.8 lb

6. Determine the strength value of the compression members

Connection capacity = 1265 lb

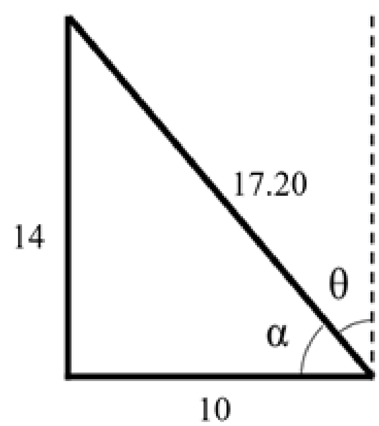
(See step 1. Capacity in tension and compression are the same)

1265 lb > 1081.8 lb ∴ member controls compression

Limit to 1/2 theoretical strength for compression values: See section 6-3.02, *Wood Cross Bracing.*

Reduced compression brace capacity = $\dfrac{1081.8\text{ lb}}{2}$ = 540.9 lb

7. Calculate the horizontal component of the strength value for the compression member



$C = 540.9 \cos\alpha = 314.4$ lb

Figure D-17-13. Geometric Components of Compression Strength Value for Bracing Ⓑ Bottom Tier

8. Calculate the total resisting capacity of the top tier of bracing

Total resisting capacity = Σ(C+T) = 314.4 + 735.3 = 1050 lb

SUMMARY

Summarize Results for All Tiers for 2% Dead Load

Tier	Horizontal Tension	Horizontal Compression	Total Capacity
A_{TOP}	900 lb	450 lb	1350 lb
A_{MID} = A_{BOTTOM}	845 lb	423 lb	1268 lb
B_{TOP}	762 lb	363 lb	1125 lb
B_{BOTTOM}	735 lb	315 lb	1050 lb

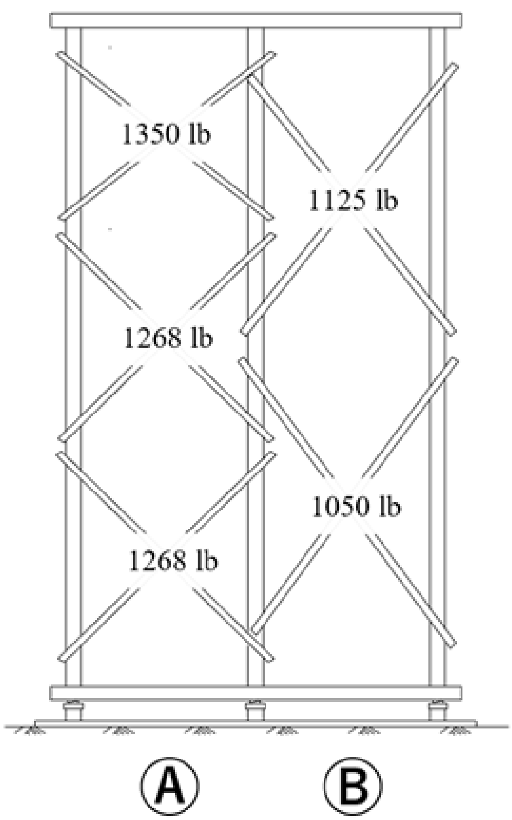


Figure D-17-14. Bracing Total Resisting Capacity for 2% Dead Load

The total resisting capacity of the bracing = the sum of the weaker pair of braces in A and the weaker pair of braces in B.

Total resisting capacity = 1268 lb + 1050 lb = 2318 lb

2318 lb (smallest total capacity) < 3500 lb (2% Dead Load)

Bracing system is inadequate for 2% Dead Load

Summarize Results for All Tiers for Wind Load

Repeat above process for wind load to calculate the Resisting Capacity, using C_D = 1.6 rather than 1.25. All other factors are the same.

The Resisting Capacity for wind load can also be derived by multiplying the resisting capacity for 2% Dead Load (above table) by the factor $\frac{1.6}{1.25}\left[\frac{C_D \text{ Wind Load}}{C_D \text{ 2\% Dead Load}}\right]$

Tier	Horizontal Tension	Horizontal Compression	Total Capacity
A_{TOP}	1152 lb	576 lb	1728 lb
$A_{MID} = A_{BOTTOM}$	1082 lb	541 lb	1623 lb
B_{TOP}	975 lb	465 lb	1440 lb
B_{BOTTOM}	941 lb	403 lb	1344 lb

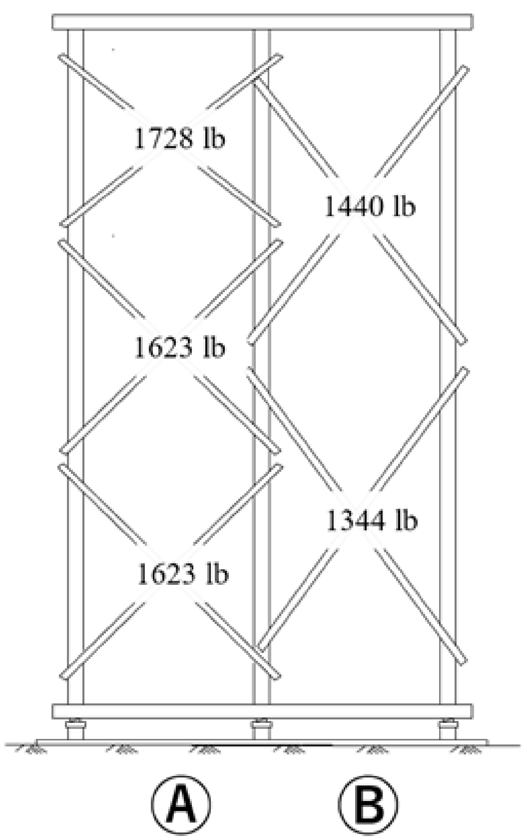


Figure D-17-15. Bracing Total Resisting Capacity for Wind Load

The total resisting capacity of the bracing = the sum of the weaker pair of braces in A and the weaker pair of braces in B.

Total resisting capacity = 1623 lb + 1344 lb = 2967 lb

2967 lb (smallest total capacity) < 3200 lb (Wind Load)

Bracing system is inadequate for Wind Load

Conclusion:
Bracing system would be adequate if bracing capacity is greater for both 2% Dead Load and Wind Load conditions.

<u>∴ Bracing System is inadequate</u>

Appendix D Example 18 – Cable Bracing – Bents

Refer to *Falsework Manual,* Section 5-5, *Cable Bracing.* This example demonstrates the adequacy of internal cable bracing of a falsework bent.

Given Information

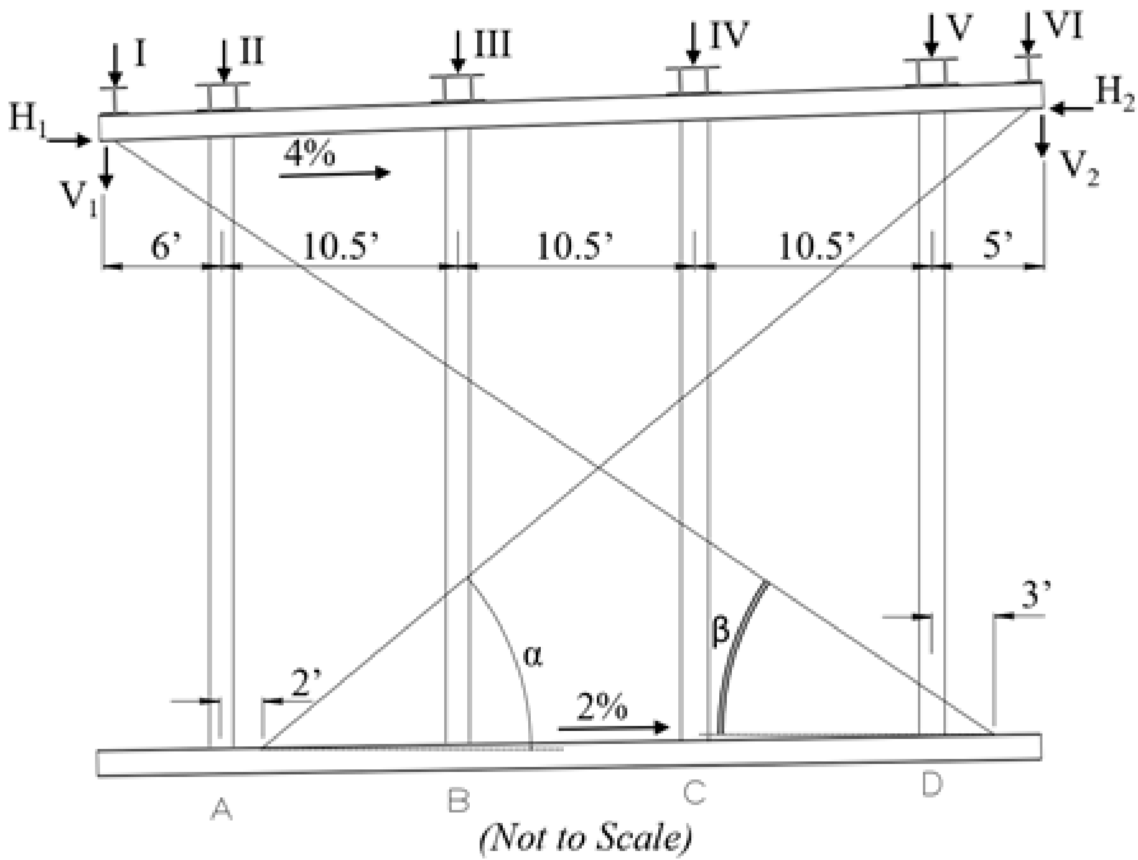


Figure D-18-1. Falsework Bent Cable Bracing

Falsework Bent:

Left Post height: 25 feet (Post A)

Cap slope: + 4%

Sill slope: + 2%

Preload: Cable 1 = 1000 lbs,

Cable 2 = 1080 lbs

Cables: 2 each, one per side

Falsework supporting a box girder bridge

Falsework bent not adjacent to traffic

Cap & Sill Beams:

W 14 x 53

$I = 541 \text{ in}^4$

Weight = 53 plf

Steel Posts:

12" Ø steel pipe

Wall thickness = 1/4"

$A = 9.23 \text{ in}^2$

$S = 26.56 \text{ in}^3$

r = 4.16 in

Cable Data from Manufacturer:

New l/2" Ø IWRC 6 x 19 wire rope

Breaking strength = 11.50 Tons

Metallic area of cable = 0.118 in^2

Cable weight = 0.46 plf

Modulus of elasticity = 13.5×10^6 psi (12.2×10^6 psi up to 20% of ultimate load)

Constructional stretch = 0.5%

Safety factor = 3

Cable clip efficiency = 80% use (Table 5-1)

Determine if the Bracing System is Adequate

Determine post heights, cable lengths, vertical and horizontal distances between each cable connection

Use geometry to find the necessary information

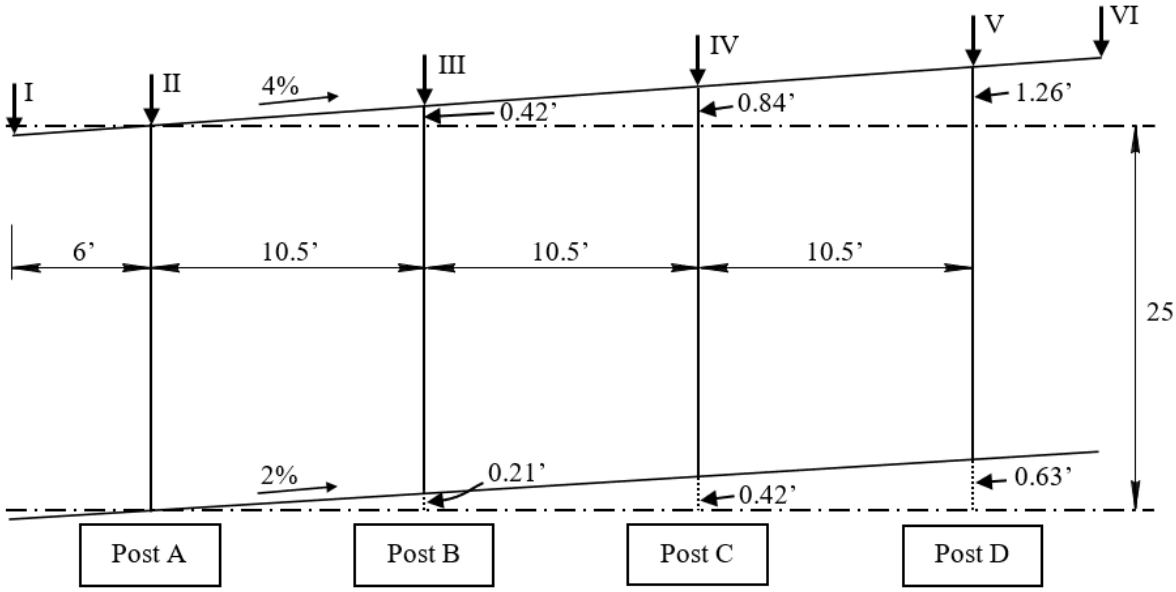


Figure D-18-2. Dimensional Analysis of Post Heights

Post Heights:

A = 25.00 ft

B = 25+0.42-0.21= 25.21 ft

C = 25+0.84-0.42= 25.42 ft

D = 25+1.26-0.63= 25.63 ft

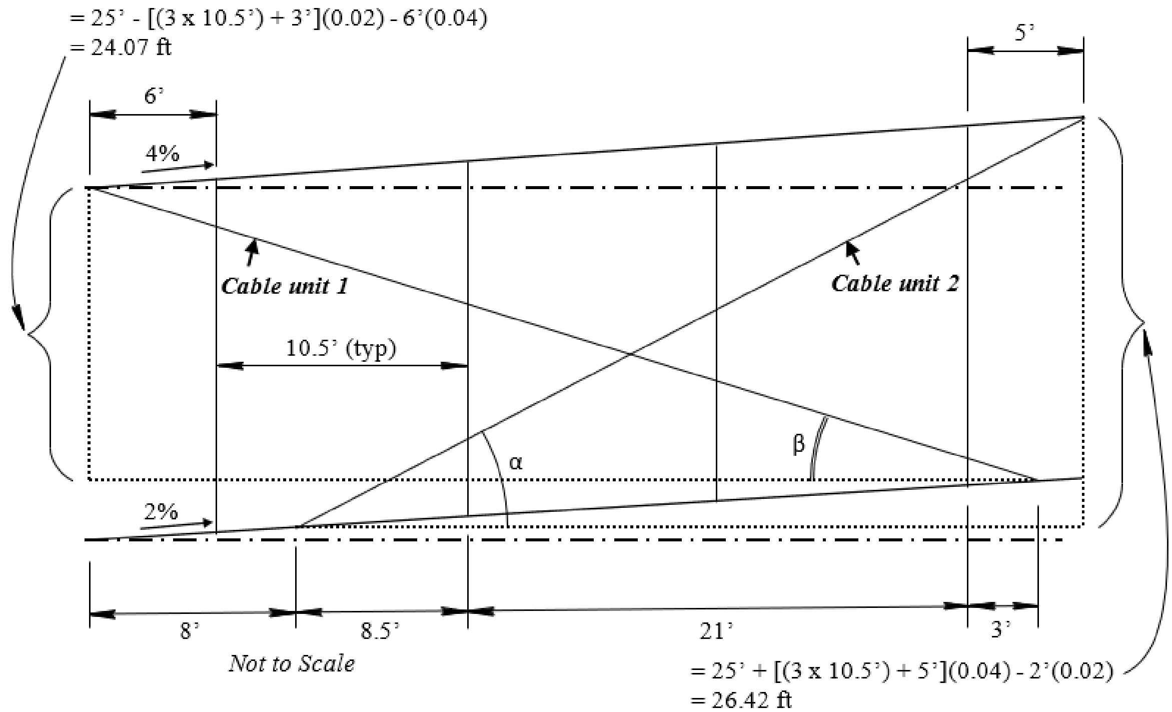


Figure D-18-3. Dimensional Analysis of Cable Bracing

Cable Angles:

Cable Unit 1: (using known post height = 25 ft)

$$\tan \beta = \frac{25 - [(3 \text{ x } 10.5) + 3](0.02) - 6(0.04)}{6+3(10.5)+3} = \frac{24.07}{40.5}$$

$\beta = 30.72°$

Cable Unit 2: (using known post height = 25 ft)

$$\tan \alpha = \frac{25 + [(3\ x\ 10.5) + 5](0.04) - 2(0.02)}{8.5+2(10.5)+5} = \frac{26.42}{34.5}$$

$\alpha = 37.44°$

Cable Lengths (assuming no drape):

$L_1 = \sqrt{24.07^2 + 40.5^2} = 47.11$ ft

$L_2 = \sqrt{26.42^2 + 34.5^2} = 43.45$ ft

Calculate the horizontal design load

Table 1 – Loads from Stringers

	Stringer Loads (kips)					
Loading Condition	**Stringer I**	**Stringer II**	**Stringer III**	**Stringer IV**	**Stringer V**	**Stringer VI**
Total DL + LL	20	73	76	90	69	19
Total DL Only	17	61	64	75	59	16
Soffit Slab & Stem DL + LL	13	51	46	55	42	11

Assume the 2% loading controls (from Table 1)

Total DL only = 17+61+64+75+59+16 = 292 kips

Horizontal load = 2% of total dead load = (292,000)(0.02) = 5840 lbs

Calculate the capacity of the cable units

The cable capacity is determined for static loading conditions by using the breaking (ultimate) strength divided by an appropriate factor of safety, in this case the safety factor = **3** (Ref. 5-5.06 *Factor of Safety*).

Cable working capacity = Strength/Safety factor = $\dfrac{(11.5\ \text{Tons})\left(2{,}000\frac{\text{lbs}}{\text{Ton}}\right)}{3}$

= 7667 lbs

Working load = (80%)(7667) = 6134 lbs (with Crosby clips efficiency applied)

Check the cable preload values

Check that the horizontal component of the Cable Unit 2 preload balances that of Cable Unit 1:

Cable Unit 1 designated preload = 1000 lbs

Preload the individual cables of Cable Unit 2 to:

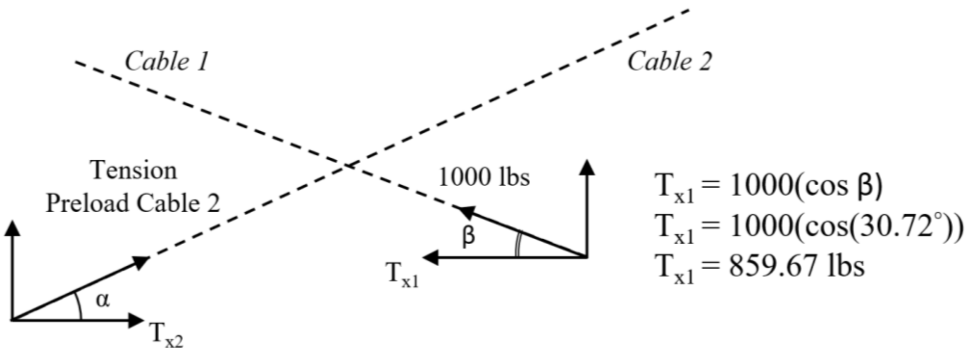


Figure D-18-4. Cable Bracing Geometry

$T_2 = \frac{859.67}{\cos(37.44)}$ = 1083 lbs ≈ 1080 lbs = Cable Unit 2 designated preload

Preload value of Cable Unit 2 balances Cable Unit 1. **OK**

Additionally, check that the cable drape after preload doesn't exceed the maximum drape:

Use the equation found in Figure 5-18, *Cable Drape Formula* to determine the distance from the chord to the loaded cable and compare to the maximum drape found in Table 5-5, *Maximum Cable Drape.*

Maximum Cable Drape for 1/2" diameter cable = 2 in

Cable Unit 1 $A = \frac{(0.46)(40.5)^2}{8(1000)(cos(30.72))}$ = 0.11 ft = 1.33 in

1.33 in < 2 in allowable **OK**

Cable Unit 2 $A = \frac{(0.46)(34.5)^2}{8(1080)(\cos(37.44))}$ = 0.079 ft = 0.96 in

0.95 in < 2 in allowable **OK**

Calculate the cable unit design loads and compare with the cable unit capacity

Use the horizontal design load to calculate the cable unit design load:

Cable Unit 1 $P = \frac{5840}{2(\cos 30.72)} = 3397 \text{ lbs} < 6134 \text{ lbs}$ **OK**

Cable Unit 2 $P = \frac{5840}{2(\cos 37.44)} = 3678 \text{ lbs} < 6134 \text{ lbs}$ **OK**

Calculate the cable unit elongations

The cable will experience two stretch conditions, elastic stretch and constructional stretch.

Elastic stretch:

Check if cable design load for both cables exceeds 20% of minimum breaking force.

20% of minimum breaking force = (0.20)(11.5 tons)(2000 lbs/ton) = 4600 lbs

Cable Unit 1 Design Load = 3397 lbs < 4600 lbs

Cable Unit 2 Design Load = 3678 lbs < 4600 lbs

Design loads of both cables do not exceed 20% minimum breaking force; therefore, use equation 5-5.09C(1)-2 to find elastic stretch (if design loads had exceeded 20% minimum breaking force, equations 5-5.09C(1)-3 and 5-5.09C(1)-4 would have been used to find the total elastic stretch).

$$\Delta = \frac{(\textbf{Cable Design Load}-\textbf{Preload})(\mathbf{L})}{\mathbf{A}\ (\mathbf{0.90E})}$$

Cable Unit 1 L=47.11 ft

Cable Unit 2 L=43.45 ft

Metallic area of cable = 0.118 in^2

Modulus of elasticity = 13.5 x 10^6 psi (12.2 x 10^6 psi up to 20% of ultimate load)

Cable Unit 1 $\Delta = \frac{[(3397-1000)](47.11)}{(0.118)(.90)(13.5 \text{ x } 10^6)} = 0.079$ ft

Cable Unit 2 $\Delta = \frac{[(3678 - 1080)](43.45)}{(0.118)(.90)(13.5 \text{ x } 10^6)}$ = 0.079 ft

Constructional Stretch:

Assume that the total constructional stretch comes out at 65% of the ultimate load and that the stretch is proportional to the load applied. Use the following formula for constructional stretch:

$\Delta_{CS} = \left(\frac{\textbf{Cable Design Load}}{\textbf{.65 Min.Breaking Force}}\right)$(Constructional Stretch)(L)

65% of minimum breaking force = (0.65)(11.5 tons)(2000 lbs/ton) = 14,950 lbs
Constructional Stretch = 0.5%

Cable Unit 1 $\Delta_{CS} = \left(\frac{3397}{14950}\right)$(0.005)(47.11) = 0.054 ft

Cable Unit 2 $\Delta_{CS} = \left(\frac{3678}{14950}\right)$(0.005)(43.45) = 0.053 ft

Total stretch:

Cable Unit 1 L (after stretch) = 47.11 + 0.079 + 0.054 = 47.24 ft

Cable Unit 2 L (after stretch) = 43.45 + 0.079 + 0.053 = 43.58 ft

Note that the effects of cap or sill bending can generally be ignored for short cantilever conditions.

Calculate the horizontal cap movement and compare with the allowable horizontal displacement

a = vertical distance between the cable connection at the cap and the point on the sill directly below it.
b = cable length before stretch
b' = cable length after stretch
c = slope distance between the point on the sill described for a, and the cable connection on the sill.

Cable Unit 1 Loaded:

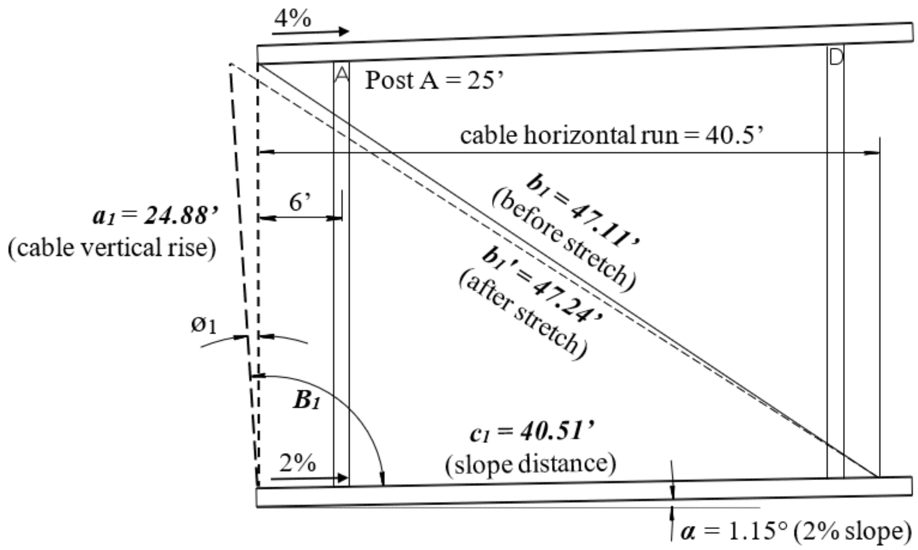


Figure D-18-5. Cap Movement with Cable Unit 1 Loaded

a_1 = 25.00 - (6)(0.04) + (6)(0.02) = 24.88 ft

$c_1 = \dfrac{40.5}{\cos\alpha} = \dfrac{40.50}{Cos\ 1.15°}$ = 40.51 ft

$\cos B_1 = \left[\dfrac{a_1{}^2 + c_1{}^2 - b_1{}'^2}{2a_1c_1}\right]$ (Law of Cosines)

$B_1 = \cos^{-1}\left[\dfrac{(24.88)^2 + (40.51)^2 - (47.24)^2}{(2)(24.88)(40.51)}\right] = 89.19^{\circ}$

$ø_1 = B_1 – (90^{\circ} – \alpha) = 89.19^{\circ} – (90^{\circ} – 1.15^{\circ}) = 0.34^{\circ}$

Horizontal deflection limit (Ref. 5-5.07) = $\dfrac{25\text{ ft}}{8} = 3.125\text{ in} \leq \dfrac{12\text{ in}}{4} = 3\text{in}$

use Δ_{max} = 3 in

Upper Cap Displacement = 24.88(sin(0.34°)) = 0.148 ft = 1.77 in

1.77 in ≤ 3 in allowable **OK**

Cable Unit 2 Loaded:

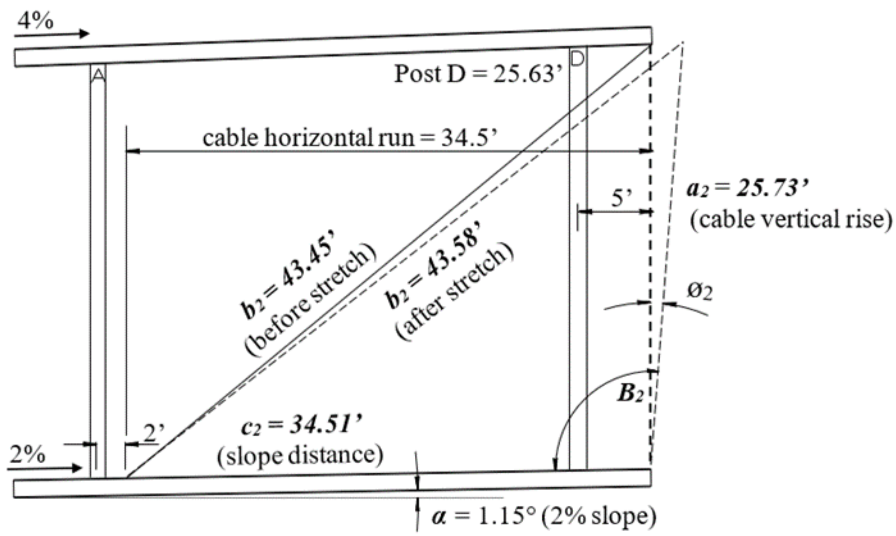


Figure D-18-6. Cap Movement with Cable Unit 2 Loaded

$a_2 = 25.63 + (5)(0.04) - (5)(0.02) = 25.73$ ft

$c_2 = \dfrac{34.5}{\cos\alpha} = \dfrac{34.5}{\cos(1.15°)} = 34.51$ ft

$\cos B_2 = \left[\dfrac{a_2{}^2 + c_2{}^2 - b_2{}'^2}{2a_2c_2}\right]$ (Law of Cosines)

$B_2 = \cos^{-1}\left[\dfrac{(25.73)^2 + (34.51)^2 - (43.58)^2}{(2)(25.73)(34.51)}\right] = 91.49°$

$ø_2 = B_2 – (90° + \alpha) = 91.49° – (90° + 1.15°) = 0.34°$

Upper Cap Displacement = 25.73 ((sin(0.34°)) = 0.153 ft. = 1.84 in.

1.84 in ≤ 3 in allowable **OK**

Determine Post Adequacy

For cable bracing systems supporting box girder structures, check for post adequacy for the two loading conditions provided in the *Standard Specifications*, Section 48-2.02B(2), *Loads*.

Check Case I:

Calculate post loads

Compute the post loads resulting from the soffit and stem reactions (Table 1) along with the vertical component of Cable 1 loading (use moment distribution or other acceptable means). Use the design horizontal load and appropriate cable angle to find the vertical component. Repeat the calculations for Cable 2 loading.

$P_{vertical}$ (Cable Unit 1) = (5,840)(tan (30.72°)) = 3470 lbs

$P_{vertical}$ (Cable Unit 2) = (5,840)(tan (37.44°)) = 4471 lbs

Table 2 Post Loads – Case I: Live Load, Soffit and Stems (No Deck Load) + Cable Loads

	Post Loads (lbs)			
Loading Condition	**Post A**	**Post B**	**Post C**	**Post D**
Case I + Cable 1 Only	79,695	33,531	50,933	59,570
Case I + Cable 2 Only	73,572	37,549	46,758	66,819

Investigate each post

$$\frac{f_a}{F_a} \leq 1$$

Where:

$f_a = \frac{P}{A}$

$F_a = 16{,}000 - 0.38\left(\frac{L}{r}\right)^2$ psi (SS 48-2.02B(3)(c) unidentified steel)

Sample calculation for stress in Post A with Cable Unit 1 loaded:

P = 79,695 lbs (from Table 2)

$f_a = \frac{P}{A} = \frac{79{,}695}{9.23} = 8634$ psi

$F_a = 16{,}000 - 0.38\left(\frac{(25)(12)}{4.16}\right)^2 = 14{,}024$ psi

$\frac{8634}{14{,}024} = 0.62 < 1$ <u>**OK**</u>

Perform the stress calculations for all four posts. Table 3 lists the results of these calculations.

Table 3 Summary of Stresses – Case I: Live Load, Soffit and Stems (No Deck Load) + Cable Loads

	Post A	**Post B**	**Post C**	**Post D**
F_a (psi)	14,024	13,990	13,957	13,923
Case I + Cable 1 Only				
f_a (psi)	8,634	3,633	5,518	6,454
Stress ratio	0.62	0.26	0.40	0.46
Case I + Cable 2 Only				
f_a (psi)	7,971	4,068	5,066	7,239
Stress ratio	0.57	0.29	0.36	0.52

The stress ratio for each post is less than 1.0; therefore, all four posts are satisfactory.

Check Case II:

Calculate post loads

Compute the post loads resulting from entire superstructure cross section reactions (Table 1) without cable loading.

Table 4 Post Loads – Case II: Live Load, Total Dead Load, (No Cable Loads)

	Post Loads (lbs)			
Loading Condition	**Post A**	**Post B**	**Post C**	**Post D**
Case II	107,527	61,827	80,645	99,262

Perform the stress calculations for all four posts. Table 5 lists the results of these calculations.

Table 5 Summary of Stresses – Case II: Live Load, Total Dead Load, (No Cable Loads)

	Post A	Post B	Post C	Post D
F_a (psi)	14,024	13,990	13,957	13,923
f_a (psi)	11,650	6,698	8,737	10,754
Stress ratio	0.83	0.48	0.63	0.77

The stress ratio for each post is less than 1.0; therefore, all four posts are satisfactory.

Appendix D Example 19 – Individual Falsework Pads – Symmetrical Loading

Refer to *Falsework Manual,* Section 8-2.06A, *Analysis of Symmetrical Pads*. This example demonstrates how to analyze individual symmetrical falsework pads.

Given Information

Timber pads:
Three 6 x 12 Rough Douglas Fir-Larch #2 (G=0.50)

Corbel:
8 x 8 Rough Douglas Fir-Larch #1 (G=0.50)

Post:
8x8 Douglas Fir-Larch #1 (G=0.50)

Allowable soil pressure = 2500 psf

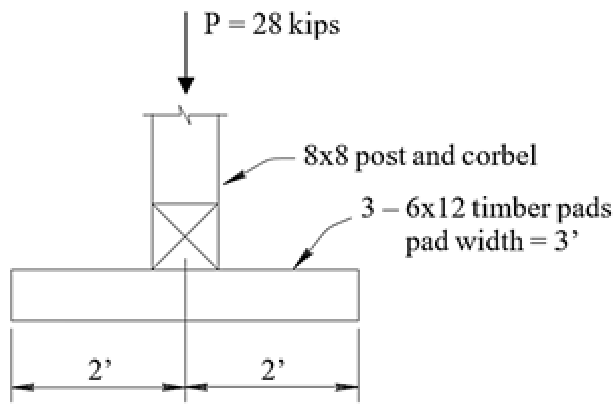


Figure D-19-1. Symmetrical Individual Pad with Single Corbel

Check Pad

1. Calculate allowable bending stress

Reference design value in bending F_b = 875 psi (NDS supplement table 4D)

Adjustment factors from NDS table 4.3.1:

C_D = 1.25	Duration Factor
C_M = 1.0	Wet Service Factor NDS table 4D (Assume < 19% moisture content)
C_t = 1.0	Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)
C_L = 1.0	Beam Stability Factor NDS 4.4.1
C_F = 1.0	Size Factor NDS Table 4D
C_{fu} = 1.0	Flat Use Factor NDS table 4D
C_i = 1.0	Incising Factor NDS 4.3.8

C_r = 1.0 Repetitive Member Factor NDS 4.3.9

Adjusted design value Fb' = Fb $(C_D)(C_M)(C_L)(C_t)(C_F)(C_i)(C_{fu})(C_r)$ = 1094 psi

2. Calculate effective length

$$L_{SYM} = \frac{1}{12}\left(\frac{8F_b'S}{1000P} + t\right) = \frac{1}{12}\left(\frac{8(1094)(216)}{1000(28)} + 8\right) = 6.29 \text{ ft}$$

$$S = \frac{bh^2}{6} = \frac{3(12)(6)^2}{6} = 216 \text{ in}^3$$

3. Find the limiting length

Compare adj. effective length and actual length 6.29 > 4.0; <u>use actual length</u>

4. Calculate soil pressure

$$\text{Soil pressure} = \frac{P}{A} = \frac{28000}{3.0(4.0)} = 2333 \text{ psf}$$

2333 psf < 2500 psf allowable **<u>OK</u>**

5. Calculate horizontal shear stress

Reference design value in shear F_v = 170 psi (NDS supplement table 4D)

Adjustment factors from NDS table 4.3.1:

C_D = 1.25	Duration Factor
C_M = 1.0	Wet Service Factor NDS table 4D (Assume < 19% moisture content)
C_t = 1.0	Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)
C_i = 1.0	Incising Factor NDS table 4.3.8

Adjusted design value F_v' = F_v $(C_D)(C_M)(C_t)(C_i)$ = 213 psi

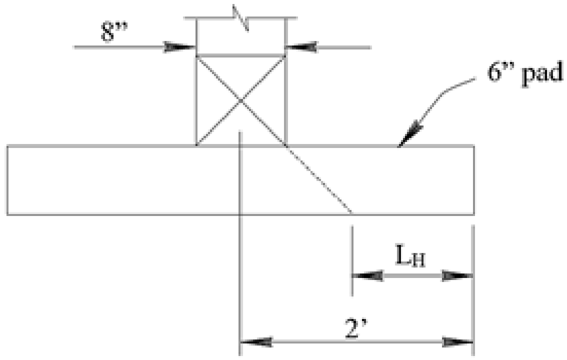


Figure D-19-2. Symmetrical Pad Shear Dimensions

$L_H = \frac{4.0}{2} - \frac{8/12}{2} - \frac{6}{12} = 1.17$ ft

V = (2333)(1.17)(3.0) = 8189 lbs.

$f_v = \frac{3V}{2A} = \frac{3(8189)}{2(6)(12)(3)} = 57$ psi

57 psi < 213 psi allowable **OK**

Check corbel

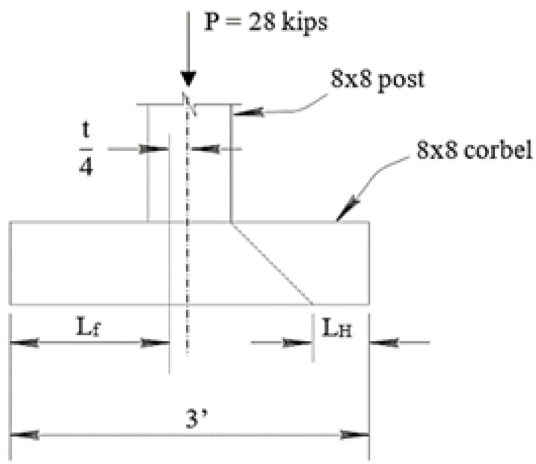


Figure D-19-3. Timber Corbel Flexure and Shear Dimensions

$W = \frac{28000}{3} = 9333$ plf

$S = \frac{bh^2}{6} = \frac{8(8)^2}{6} = 85.3\ in^3$

$L_H = \frac{3.0}{2} - \frac{8/12}{2} - \frac{8}{12} = 0.50$ ft

$L_f = \frac{3.0}{2} - \frac{8/12}{4} = 1.33$ ft

1. Calculate compression perpendicular to grain

Reference design value in shear $F_{c\perp}$ = 625 psi (NDS supplement table 4D)

Adjustment factors from NDS table 4.3.1:

$C_M = 1.0$	Wet Service Factor NDS table 4D (Assume < 19% moisture content)
$C_t = 1.0$	Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)
$C_i = 1.0$	Incising Factor NDS table 4.3.8
$C_b = 1.0$	Bearing Area Factor NDS 3.10.4

Adjusted design value $F_{c\perp}' = F_{c\perp}(C_M)(C_t)(C_i)(C_b) = 625$ psi

$$f_c = \frac{P}{A} = \frac{28000}{8(8)} = 438 \text{ psi}$$

438 psi < 625 psi allowable **OK**

2. Calculate horizontal shear stress

V = 0.5 (9333) = 4667 lbs

$$f_v = \frac{3V}{2A} = \frac{3(4667)}{2(64)} = 109 \text{ psi}$$

109 psi < 213 psi allowable **OK**

3. Calculate bending stress

$$M = \frac{WL^2}{2} = \frac{(9333)(1.33)^2}{2} = 8255 \text{ ft-lbs}$$

$$f_b = \frac{M}{S} = \frac{8255(12)}{85.3} = 1161 \text{ psi}$$

Reference design value in bending $F_b = 1350$ psi (NDS supplement table 4D)

Adjusted design value $F_b' = F_b(C_D)(C_M)(C_L)(C_t)(C_F)(C_i)(C_{fu})(C_r) = 1688$ psi (see “Pad Check” step 1 for adjustment factors)

1161 psi < F_b = 1688 psi allowable **OK**

Appendix D Example 20 – Individual Falsework Pads – Asymmetrical Loading

Refer to *Falsework Manual,* Section 8-2.06B, *Analysis of Asymmetrical Pads*. This example demonstrates how to analyze individual asymmetrical falsework pads.

Given Information

Timber pads:
Three 6x16 Rough Douglas Fir-Larch #2 (G=0.50)

Corbel:
12x12 Rough Douglas Fir-Larch #1 (G=0.50)

Post:
12x12 Douglas Fir-Larch #1 (G=0.50)

Allowable soil pressure = 3500 psf

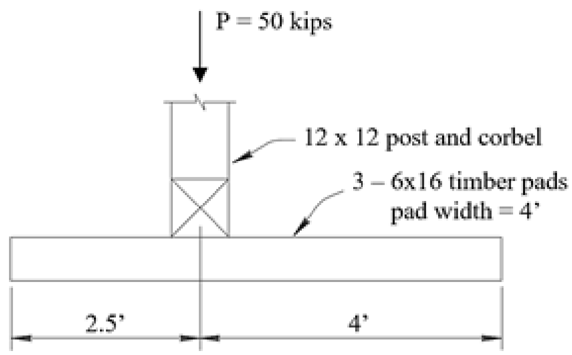


Figure D-20-1. Asymmetrical Individual Pad with Single Corbel

Check Pad

1. **Calculate allowable bending stress**

   Reference design value in bending F_b = 875 psi (NDS supplement table 4D)

   Adjustment factors from NDS table 4.3.1:

C_D = 1.25	Duration Factor
C_M = 1.0	Wet Service Factor NDS table 4D (Assume < 19% moisture content)
C_t = 1.0	Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)
C_L = 1.0	Beam Stability Factor NDS 4.4.1
C_F = 1.0	Size Factor NDS Table 4D
C_{fu} = 1.0	Flat Use Factor NDS table 4D
C_i = 1.0	Incising Factor NDS 4.3.8
C_r = 1.0	Repetitive Member Factor NDS 4.3.9

Adjusted design value $F_b' = F_b\,(C_D)(C_M)(C_t)(C_L)(C_F)(C_i)(C_{fu})(C_r)$ = 1094 psi

2. Calculate adjusted effective length

$L_e = L_{SYM} = \frac{1}{12}\left(\frac{8F_b'S}{1000P} + t\right) = \frac{1}{12}\left(\frac{8(1094)(288)}{1000(50)} + 12\right)$ = 5.20 ft

$S = \frac{bh^2}{6} = \frac{3(16)(6)^2}{6}$ = 288 in^3

3. Find limiting length on short side

Compare $\frac{1}{2}$ of effective length and actual length

$\frac{1}{2}$ (5.20) = 2.60 > 2.50; Pad length (L_1) = 2.50 ft

4. Calculate limiting length on long side

Compare $\frac{1}{2}$ of effective length and actual length

$\frac{1}{2}$ (5.20) = 2.60 < 4.0; Pad length (L_2) = 2.60 ft

5. Calculate soil pressure

Bearing length = $L_1 + L_2$ = 2.50 + 2.60 = 5.10 ft

Soil pressure = $\frac{P}{A} = \frac{50000}{4(5.10)}$ = 2451 psf

2451 psf ≤ 3500 psf allowable **OK**

6. Calculate horizontal shear on the long side

Reference design value in shear F_v = 170 psi (NDS supplement table 4D)

Adjustment factors from NDS table 4.3.1:

C_D = 1.25 Duration Factor

C_M = 1.0 Wet Service Factor NDS table 4D (Assume < 19% moisture content)

C_t = 1.0 Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)

C_i = 1.0 Incising Factor NDS table 4.3.8

Adjusted design value $F_v' = F_v (C_D)(C_M)(C_t)(C_i)$ = 213 psi

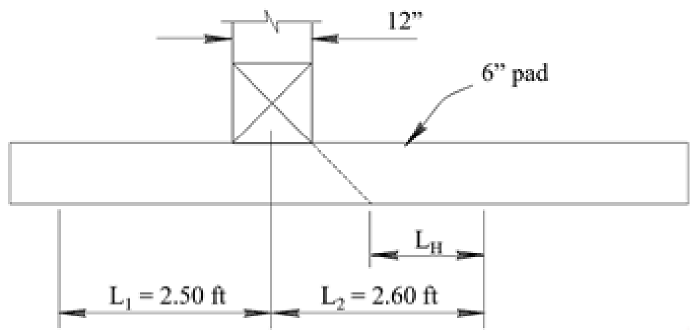


Figure D-20-2. Asymmetrical Pad Shear Dimensions

$$L_H = 2.60 - \frac{12/12}{2} - \frac{6}{12} = 1.50 \text{ ft}$$

$$V = (2451)(1.60)(4.0) = 15686 \text{ lb}$$

$$f_v = \frac{3V}{2A} = \frac{3(15686)}{2(6)(16)(3)} = 82 \text{ psi}$$

82 psi < 213 allowable **OK**

Check Corbel

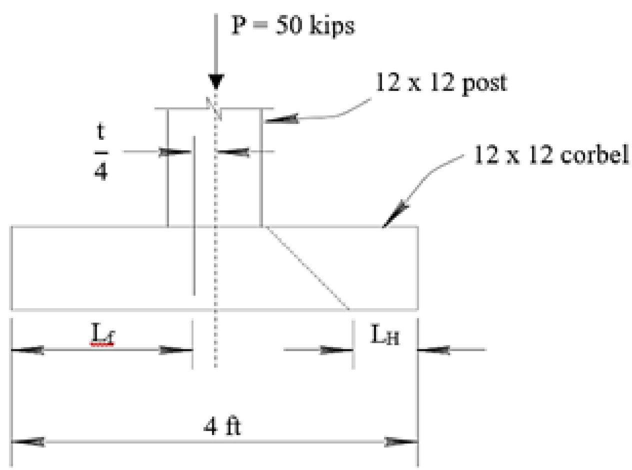


Figure D-20-3. Timber Corbel Flexure and Shear Dimensions

$$W = \frac{50000}{4} = 12500 \text{ plf}$$

$$S = \frac{bh^2}{6} = \frac{12(12)^2}{6} = 288 \text{ in}^3$$

$$L_f = \frac{4.0}{2} - \frac{12/12}{4} = 1.75 \text{ ft}$$

$$L_H = \frac{4.0}{2} - \frac{12/12}{2} - \frac{12}{12} = 0.5 \text{ ft}$$

1. Calculate compression perpendicular to grain

Reference design value in shear $F_{c\perp}$ = 625 psi (NDS supplement table 4D)
Adjustment factors from NDS table 4.3.1:

C_M = 1.0 Wet Service Factor NDS table 4D (Assume < 19% moisture content)

C_t = 1.0 Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)

C_i = 1.0 Incising Factor NDS table 4.3.8

C_b = 1.0 Bearing Area Factor NDS 3.10.4

Adjusted design value $F_{c\perp}' = F_{c\perp}(C_M)(C_t)(C_i)(C_b)$ = 625 psi

$f_c = \frac{P}{A} = \frac{50000}{12 \text{ x } 12} = 347$ psi

347 psi < 625 allowable **OK**

2. Calculate stress due to horizontal shear

V = (12500)(0.5) = 6250 lbs

$f_v = \frac{3V}{2A} = \frac{3(6250)}{2(12)(12)} = 65$ psi

65 psi < 213 allowable **OK**

3. Calculate bending stress

$M = \frac{WL^2}{2} = \frac{(12500)(1.75)^2}{2}$ 19141 ft-lbs

$f_b = \frac{M}{S} = \frac{(19141)(12)}{288} = 798$ psi

Reference design value in bending F_b = 1350 psi (NDS supplement table 4D)

Adjusted design value $F_b' = F_b\,(C_D)(C_M)(C_t)(C_L)(C_F)(C_i)(C_{fu})(C_r)$ = 1688 psi (see “Pad Check” step 1 for adjustment factors)

798 psi < 1688 psi allowable **OK**

Appendix D Example 21 – Continuous Pads – Individual Corbels

Refer to *Falsework Manual,* Section 8-2.04, *Continuous Pad with Single Corbel.* This example demonstrates how to analyze a continuous falsework pad with single corbels.

Given Information

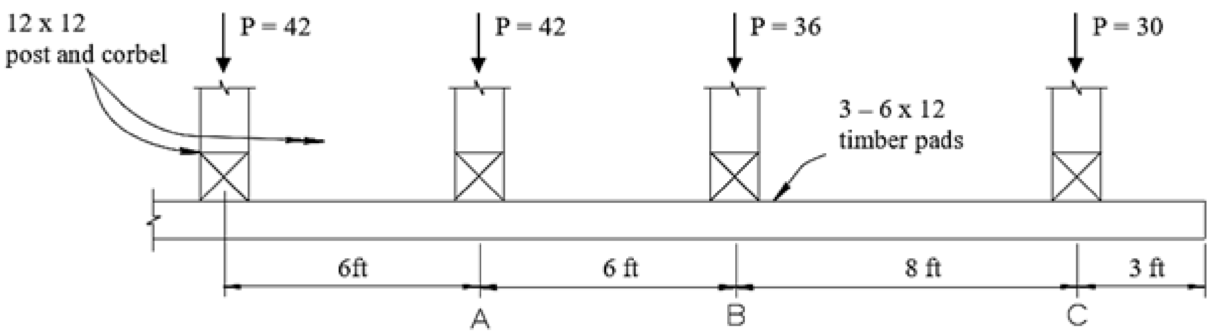


Figure D-21-1. Continuous Pad with Single Corbels

Timber pads:

Three 6 x 12 Rough Douglas Fir-Larch #2 (G=0.50)

Corbel:

12 x 12 Rough Douglas Fir-Larch #1 (G=0.50)

Post:

12 x 12 Douglas Fir-Larch #1 (G=0.50)

Allowable soil pressure = 3000 psf

Post A:

Check Pad

1. **Calculate allowable bending stress**

   Reference design value in bending F_b = 875 psi (NDS supplement table 4D)

Adjustment factors from NDS table 4.3.1:

C_D = 1.25	Duration Factor
C_M = 1.0	Wet Service Factor NDS table 4D (Assume < 19% moisture content)
C_t = 1.0	Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)
C_L = 1.0	Beam Stability Factor NDS 4.4.1
C_F = 1.0	Size Factor NDS Table 4D
C_{fu} = 1.0	Flat Use Factor NDS table 4D
C_i = 1.0	Incising Factor NDS 4.3.8
C_r = 1.0	Repetitive Member Factor NDS 4.3.9

Adjusted design value Fb' = Fb $(C_D)(C_M)(C_t)(C_L)(C_F)(C_i)(C_{fu})(C_r)$ = 1094 psi

2. Calculate effective length

$$L_e = L_{SYM} = \frac{1}{12}\left(\frac{8F_b'S}{1000P} + t\right) = \frac{1}{12}\left(\frac{8(1094)(216)}{1000(42)} + 12\right) = 4.75 \text{ ft}$$

$$S = \frac{bh^2}{6} = \frac{3(12)(6)^2}{6} = 216 \text{ in}^3$$

3. Find the limiting length

Compare effective length and post spacing 4.75 ft < 6.0 ft; <u>Use effective length</u>

4. Calculate soil pressure

$$\text{Soil pressure} = \frac{P}{A} = \frac{42000}{3.0(4.75)} = 2947 \text{ psf}$$

2947 psf < 3000 psf allowable **<u>OK</u>**

5. Calculate horizontal shear stress

Reference design value in shear F_v = 170 psi (NDS supplement table 4D)

Adjustment factors from NDS table 4.3.1:

C_D = 1.25 Duration Factor

C_M = 1.0	Wet Service Factor NDS table 4D (Assume < 19% moisture content)
C_t = 1.0	Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)
C_i = 1.0	Incising Factor NDS table 4.3.8

Adjusted design value $F_v' = F_v (C_D)(C_M)(C_t)(C_i)$ = 213 psi

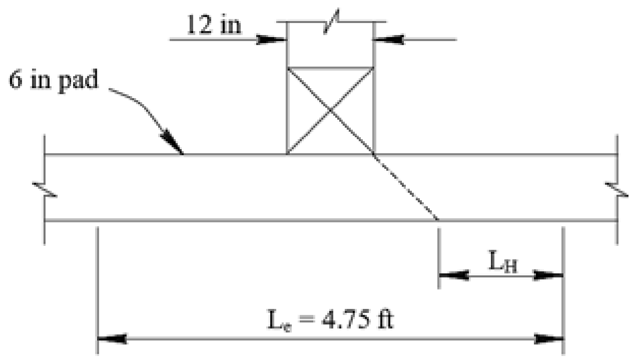


Figure D-21-2. Post A Continuous Pad Shear Dimension

$L_H = \frac{4.75}{2} - \frac{12/12}{2} - \frac{6}{12}$ = 1.38 ft

V = (2947)(1.38)(3.0) = 12201 lbs

$f_v = \frac{3V}{2A} = \frac{3(12201)}{2(6)(12)(3)}$ = 85 psi

85 psi < 213 psi allowable **OK**

Check corbel

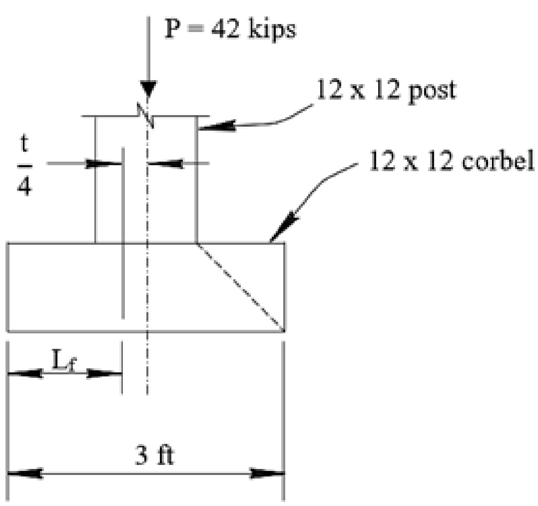


Figure D-21-3. Post A Timber Corbel Flexure and Shear Dimensions

$W = \frac{42000}{3}$ = 14000 plf

$S = \frac{bh^2}{b} = \frac{12(12)^2}{6}$ = 288 in^3

A = bh = 12(12) = 144 in^2

$L_f = \frac{3.0}{2} - \frac{12/12}{4}$ = 1.25 ft

$L_H = \frac{3.0}{2} - \frac{12/12}{2} - \frac{12}{12}$ = 0 ft

1. Calculate compression perpendicular to grain

Reference design value $F_{c\perp}$ = 625 psi

Adjustment factors from NDS table 4.3.1:

C_M = 1.0 Wet Service Factor NDS table 4D (Assume < 19% moisture content)

C_t = 1.0 Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)

C_i = 1.0 Incising Factor NDS table 4.3.8

C_b = 1.0 Bearing Area Factor NDS 3.10.4

Adjusted design value $F_{c\perp}' = F_{c\perp}(C_M)(C_t)(C_i)(C_b)$ = 625 psi

$f_c = \frac{P}{A} = \frac{42000}{144} = 292$ psi

292 psi < 625 psi allowable **OK**

2. **Calculate horizontal shear stress**

V = 0(14000) = 0 lbs.

$f_v = \frac{3V}{2A}$ = 0 psi < 213 psi **OK**

3. **Calculate bending stress**

$M = \frac{WL^2}{2} = \frac{(14000)(1.25)^2}{2}$ = 10938 ft-lbs

$f_b = \frac{M}{S} = \frac{10938(12)}{288}$ = 456 psi

Reference design value in bending F_b = 1350 psi (NDS supplement table 4D)

Adjusted design value $F_b' = F_b(C_D)(C_M)(C_t)(C_L)(C_F)(C_i)(C_{fu})(C_r)$ = 1688 psi (see "Pad Check" step 1 for adjustment factors)

f_b = 456 psi < F_b = 1688 psi allowable **OK**

Post B:

Check Pad

1. **Calculate effective length of pad short side**

$L_{SYM} = \frac{1}{12}\left(\frac{8F_b'S}{1000P} + t\right) = \frac{1}{12}\left(\frac{8(1094)(216)}{1000(36)} + 12\right)$ = 5.38 ft

$$S = \frac{bh^2}{6} = \frac{3(12)(6)^2}{6} = 216 \text{ in}^3$$

2. Find limiting length on short side

Compare $\frac{1}{2}$ of effective length and $\frac{1}{2}$ post spacing

$\frac{1}{2}$ (5.38 ft) = 2.69 ft

$$\frac{\text{post spacing}}{2} = \frac{6.00}{2} = 3.00$$

L_1 = 2.69 ft (min. from above)

3. Find limiting length on long side

Compare $\frac{1}{2}$ of effective length and $\frac{1}{2}$ post spacing

$\frac{1}{2}$ (5.38) = 2.69 ft

$$\frac{\text{post spacing}}{2} = \frac{8.0}{2} = 4.00 \text{ ft}$$

L_2 = 2.69 ft (min. from above)

4. Calculate soil pressure

Bearing length = $L_1 + L_2$ = 2.69 + 2.69 = 5.38 ft

Soil pressure = $\frac{P}{A} = \frac{36000}{3(5.38)}$ = 2230 psf

2230 psf < 3000 allowable **OK**

5. Calculate horizontal shear in pad on long side

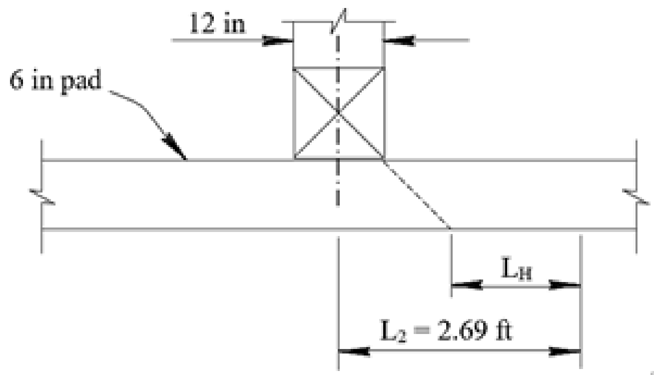


Figure D-21-4. Post B Continuous Pad Shear Dimension

$L_H = 2.69 - 0.5 - 0.5 = 1.69$ ft

$V = (2230)(1.69)(3.0) = 11306$ lbs.

$f_v = \frac{3V}{2A} = \frac{3(11306)}{2(6)(12)(3)} = 79$ psi < 213 psi

OK

Check Corbel

Corbel is **OK** by inspection (Post load at B < post load at A)

Post C:

Check Pad

1. **Calculate effective length of pad**

$$L_{SYM} = \frac{1}{12}\left(\frac{8F_b'S}{1000P} + t\right) = \frac{1}{12}\left(\frac{8(1094)(216)}{1000(30)} + 12\right) = 6.25 \text{ ft}$$

2. **Find limiting length on outside of post**

Compare $\frac{1}{2}$ of effective length and edge distance

$\frac{1}{2}(6.25) = 3.13$ ft

Edge distance = 3.00 ft

$L_1 = 3.0$ ft (min. from above)

3. **Find limiting length on inside of post**

Compare $\frac{1}{2}$ of effective length and $\frac{1}{2}$ post spacing

$\frac{1}{2}(6.25) = 3.13$ ft

$\frac{\text{post spacing}}{2} = \frac{8.00}{2}$ = 4.00 ft

L_2 = 3.13 ft (min. from above)

4. **Calculate soil pressure**

Bearing length = $L_1 + L_2$ = 3.00 + 3.13 = 6.13 ft

Soil pressure = $\frac{P}{A} = \frac{30000}{3(6.13)}$ = 1631 psf

1631 psf < 3000 psf allowable **OK**

5. **Calculate horizontal shear stress on long side**

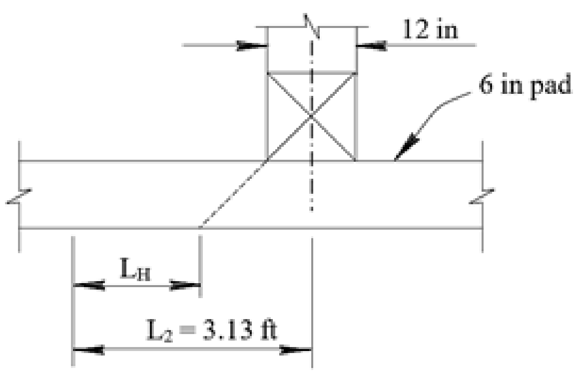


Figure D-21-5. Post C Continuous Pad Shear Dimension

$L_H = 3.13 - \frac{12/12}{2} - \frac{6}{12}$ = 2.13 ft

V = (1631)(2.13)(3.00) = 10422 lbs

$f_v = \frac{3V}{2A} = \frac{3(10422)}{2(6)(12)(3)}$ = 72 psi

72 psi < 213 psi allowable **OK**

Check Corbel

Corbel is **OK** by inspection (Post load at C < post load at A)

Appendix D Example 22 – Continuous Pads – Two or More Corbels

Refer to *Falsework Manual,* Section 8-2.05, *Continuous Pad with Two or More Corbels.* This example demonstrates how to analyze a continuous falsework pad with multiple corbels.

Given Information

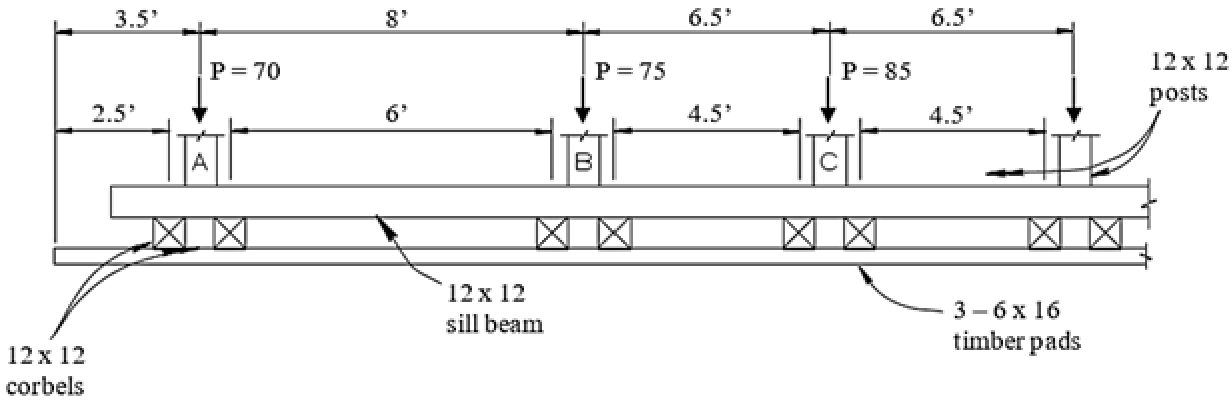


Figure D-22-1. Continuous Pad with Two Corbels

Timber pads:

Three 6 x 16 Rough Douglas Fir-Larch #2 (G=0.50)

Corbel:

12 x 12 Rough Douglas Fir-Larch #1 (G=0.50)

Post:

12 x 12 Douglas Fir-Larch #1 (G=0.50)

Allowable soil pressure = 4000 psf

Post A:

Check Pad

1. **Calculate allowable bending stress**

   Reference design value in bending F_b = 875 psi (NDS supplement table 4D)

Adjustment factors from NDS table 4.3.1:

C_D = 1.25	Duration Factor
C_M = 1.0	Wet Service Factor NDS table 4D (Assume < 19% moisture content)
C_t = 1.0	Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)
C_L = 1.0	Beam Stability Factor NDS 4.4.1
C_F = 1.0	Size Factor NDS Table 4D
C_{fu} = 1.0	Flat Use Factor NDS table 4D
C_i = 1.0	Incising Factor NDS 4.3.8
C_r = 1.0	Repetitive Member Factor NDS 4.3.9

Adjusted design value $F_b' = F_b\,(C_D)(C_M)(C_t)(C_L)(C_F)(C_i)(C_{fu})(C_r)$ = 1094 psi

2. Calculate effective length of pad

$$L_e = L_{SYM} = \frac{1}{12}\left(\frac{8F_b'S}{1000P} + t\right) = \frac{1}{12}\left(\frac{8(1094)(288)}{1000(70)} + 12\right) = 4.00 \text{ ft}$$

$$S = \frac{bh^2}{6} = \frac{3(16)(6)^2}{6} = 288 \text{ in}^3$$

3. Find limiting length of outside of post

Compare $\frac{1}{2}$ of effective length and edge distance

$\frac{1}{2}(4.0) = 2.00$ ft

Edge distance = 2.50 ft

L_1 = 2 ft (min. from above)

4. Find limiting length on inside of post

Compare $\frac{1}{2}$ of effective length and $\frac{1}{2}$ corbel spacing

$\frac{1}{2}L_e = \frac{1}{2}(4.0) = 2.00$ ft

$\frac{1}{2}$ (corbel spacing) = $\frac{1}{2}$ (6.0) = 3.00 ft

L_2 = 2.00 ft (min. from above)

5. **Calculate soil pressure**

Bearing length = L_1 + m + L_2 = 2.00 + 2.00 + 2.00 = 6.00 ft

Soil pressure = $\frac{P}{A} = \frac{70000}{4(6.0)} = 2917$ psf

2917 < 4000 allowable **<u>OK</u>**

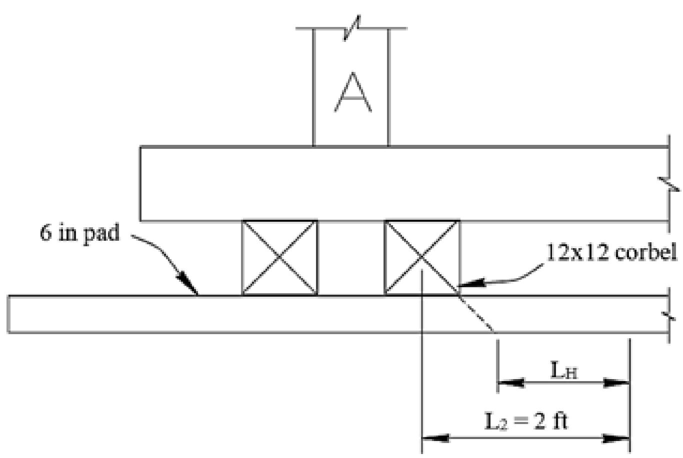
6. **Calculate horizontal shear on long side of pad**

Reference design value in shear F_v = 170 psi (NDS supplement table 4D)

Adjustment factors from NDS table 4.3.1:

C_D = 1.25	Duration Factor
C_M = 1.0	Wet Service Factor NDS table 4D (Assume < 19% moisture content)
C_t = 1.0	Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)
C_i = 1.0	Incising Factor NDS table 4.3.8

Adjusted design value $F_v' = F_v (C_D)(C_M)(C_t)(C_i)$ = 213 psi



$L_H = 2.00 - \frac{12/12}{2} - \frac{6}{12} = 1.00$ ft

V = (2916)(1.0)(4.00) = 11667 lb

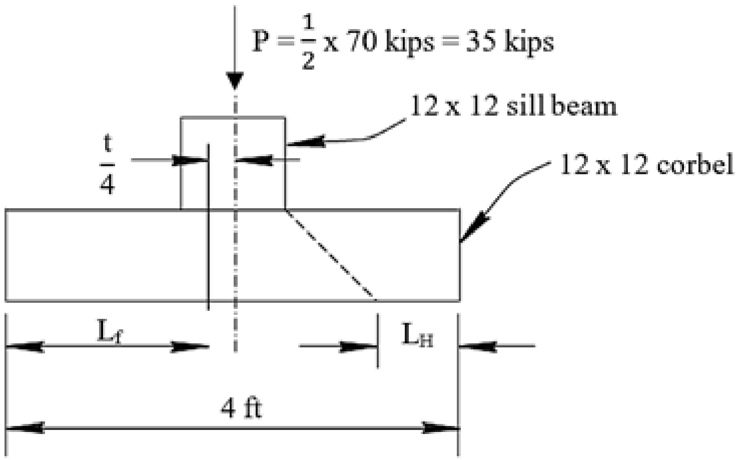
$f_v = \frac{3V}{2A} = \frac{3(11667)}{2(6)(16)(3)} = 61$ psi

61 psi < 213 psi allowable **<u>OK</u>**

Figure D-22-2. Exterior Post A Continuous Pad Shear Dimension

Check Corbels

Assume total vertical load is distributed equally to the two corbels.



$$W = \frac{35000}{4} = 8750 \text{ plf}$$

$$S = \frac{bh^2}{6} = \frac{(12(12))^2}{6} = 288 \text{ in}^3$$

$$L_f = \frac{4.0}{2} - \frac{12/12}{4} = 1.75 \text{ ft}$$

$$L_H = \frac{4.0}{2} - \frac{12/12}{2} - \frac{12}{12} = 0.5 \text{ ft}$$

Figure D-22-3. Post A Timber Corbel Flexure and Shear Dimensions

1. **Calculate compression perpendicular to grain**

   Reference design value $F_{c\perp}$ = 625 psi

   Adjustment factors from NDS table 4.3.1:

   C_M = 1.0 Wet Service Factor NDS table 4D (Assume < 19% moisture content)
   C_t = 1.0 Temperature Factor NDS table 2.3.3 (Temp up to 100ºF)
   C_i = 1.0 Incising Factor NDS table 4.3.8
   C_b = 1.0 Bearing Area Factor NDS 3.10.4

   Adjusted design value $F_{c\perp}' = F_{c\perp}(C_M)(C_t)(C_i)(C_b)$ = 625 psi

   $$F_c = \frac{P}{A} = \frac{35000}{12(12)} = 243 \text{ psi}$$

   243 psi < 625 psi allow **OK**

2. **Calculate horizontal shear stress in corbel**

   V = 0.5(8750) = 4375 lb

$f_v = \frac{3V}{2A} - \frac{3(4375)}{2(144)}$ = 46 psi

46 psi < 213 psi allow **OK**

3. **Calculate bending stress**

$M = \frac{WL^2}{2} = \frac{(8750)(1.75)^2}{2}$ = 13398 ft-lb

$f_b = \frac{M}{S} = \frac{(13398)(12)}{288}$ = 558 psi

Reference design value in bending F_b = 1350 psi (NDS supplement table 4D)

Adjusted design value F_b' = F_b $(C_D)(C_M)(C_t)(C_L)(C_F)(C_i)(C_{fu})(C_r)$ = 1688 psi (see "Pad Check" step 1 for adjustment factors)

558 psi < 1688 psi allowable **OK**

Post B:

Check Pad

1. **Calculate effective length of pad**

$L_e = L_{SYM} = \frac{1}{12}\left(\frac{8F_b'S}{1000P} + t\right) = \frac{1}{12}\left(\frac{8(1094)(288)}{1000(75)} + 12\right)$ = 3.80 ft

$S = \frac{bh^2}{6} = \frac{3(16)(6)^2}{6}$ = 288 in^3

2. **Find limiting length of short (right) side**

Compare $\frac{1}{2}$ of effective length and $\frac{1}{2}$ corbel spacing

$\frac{1}{2}(3.80)$ =1.90 ft

$\frac{1}{2}$ (corbel spacing) = $\frac{1}{2}$ (4.50) = 2.25 ft

L_1 = 1.90 ft (min. from above)

3. Find limiting length on long side

Compare $\frac{1}{2}$ of effective length and $\frac{1}{2}$ corbel spacing

$\frac{1}{2}\,(3.80)$ = 1.90 ft

$\frac{1}{2}$ (corbel spacing) = $\frac{1}{2}$ (6.00) = 3.00 ft

L_2 = 1.90 ft (min. from above)

4. Calculate soil pressure

Bearing length = $L_1 + m + L_2$ = 1.90 + 2.00 + 1.90 = 5.80 ft

Soil pressure= $\frac{P}{A} = \frac{75000}{4(5.80)}$ = 3233 psf

3233 psf < 4000 psf allowable **OK**

5. Calculate horizontal shear stress on long side of pad

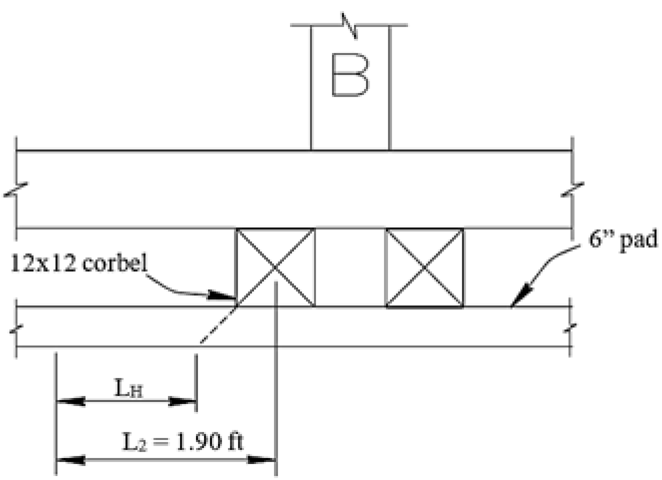


Figure D-22-4. Interior Post B Continuous Pad Shear Dimension

$L_H = 1.90 - \frac{12/12}{2} - \frac{6}{12} = 0.90$ ft

V = 4(3233)(0.90) = 11639 lb

$f_v = \frac{3V}{2A} = \frac{3(11639)}{2(6)(16)(3)}$ = 61 psi

61 psi < 213 psi allowable **OK**

Check Corbels

Post B corbel is same as Post A corbel; therefore, stress is proportional to the applied load.

$f_c = \frac{75}{70}(243) = 260$ psi < 625 psi allowable

$f_v = \frac{75}{70}(46) = 49$ psi < 213 psi allowable

$f_b = \frac{75}{70}(558) = 598$ psi < 1688 psi allowable **OK**

Post C:

Check Pad

1. **Calculate effective length of pad**

$$L_e = L_{SYM} = \frac{1}{12}\left(\frac{8F_b^{'}S}{1000P} + t\right) = \frac{1}{12}\left(\frac{8(1094)(288)}{1000(85)} + 12\right) = 3.47 \text{ ft}$$

2. **Find limiting length**

Compare $\frac{1}{2}$ of effective length and $\frac{1}{2}$ corbel spacing

$\frac{1}{2}(3.47) = 1.74$ ft

$\frac{1}{2}$ (corbel spacing) $= \frac{1}{2}(4.5) = 2.25$ ft

$L_1 = L_2 = 1.74$ ft (min. from above)

3. **Calculate soil pressure**

Bearing length $= L_1 + m + L_2 = 1.74 + 2.00 + 1.74 = 5.48$ ft

Soil pressure $= \frac{P}{A} = \frac{85000}{4(5.48)} = 3878$ psf

3878 psf < 4000 psf allowable **OK**

4. **Calculate horizontal shear stress**

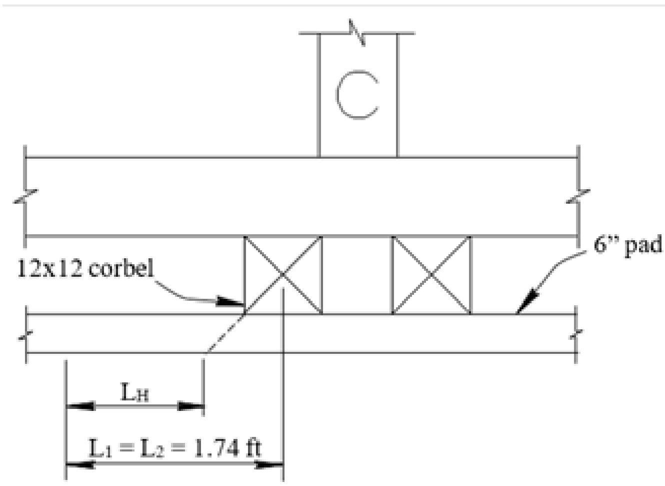


Figure D-22-5. Interior Post C Continuous Pad Shear Dimension

$L_H = 1.74 - \frac{12/12}{2} - \frac{6}{12} = 0.74$ ft

$V = 4(3878)(0.74) = 11479$ lb

$f_v = \frac{3V}{2A} = \frac{3(11479)}{2(6)(16)(3)} = 60$ psi

60 psi < 213 psi allowable **OK**

Check Corbels

Post C corbel is same as Post A corbel; therefore, stress is proportional to the applied load.

$f_c = \frac{85}{70}(243) = 295$ psi < 625 psi allowable

$f_v = \frac{85}{70}(46) = 56$ psi < 213 psi allowable

$f_b = \frac{85}{70}(558) = 678$ psi < 1688 psi allowable **OK**

Appendix D Example 23 – Soil Bearing Load Test

Refer to *Falsework Manual,* Section -8-4, S*oil Load Tests and Soil Bearing Values.* This example demonstrates procedure for determining soil bearing capacity using a static load test.

Determine the allowable bearing capacity

Contractor’s pad size: 10 ft x 10 ft

Area = 100 ft^2

Perimeter = 40 ft

Contractor's proposed pad settlement: 1/2 in

Use two test pads:

Smaller test pad:

Dimensions: 2 ft x 2 ft

Area = 4 ft^2

Perimeter = 8 ft

Load = 25,200 lb at 1/2-in settlement

Larger test pad:

Dimensions: 3 ft x 3 ft

Area = 9 ft^2

Perimeter = 12 ft

Load = 39700 lb at 1/2-in settlement

Test Summary					
Pad	Pad Area	Pad Perimeter	Total Load	Settlement	Formula
	A	P	W	S	W = An + Pm
	(ft^2)	(ft)	(lb)	(in)	
Smaller	4	8	25,200	0.5	25,200 = 4n + 8m
Larger	9	12	39,700	0.5	39,700 = 9n + 12m

Solving for (m) and (n) units by dimensional analysis:

m ≈ 2,833 plf
n ≈ 633 psf

For the actual footing:

$x = P/A = 40 \text{ ft} / 100 \text{ ft}^2 = 0.4 / \text{ft}$

Substitute (m), (n) and (x) into the equation (p = mx + n)

The allowable soil bearing value (p) for the contractor's pad is:

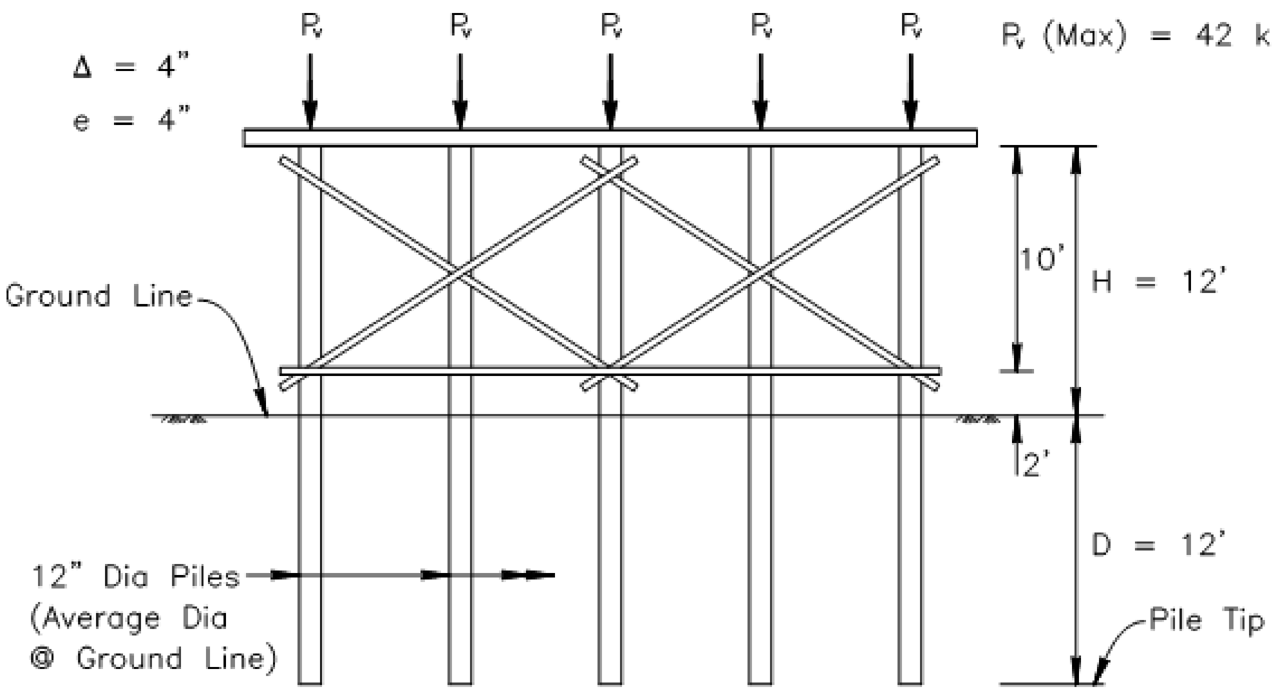
p ≈ (2,833 plf)(0.4 / ft) + 633 psf ≈ 1,766 psf

Appendix D Example 24 – Timber Pile Bents – Type I Bent

Refer to *Falsework Manual,* Section 8-6.05, *Analysis of Timber Pile Bents.*

Occasionally pile foundations will be used for falsework systems due to poor soil conditions, having to traverse over water, and to mitigate differential settlement. As-built conditions of the driven piles will dictate the bent capacity to resist horizontal loads. Type I falsework bents are analyzed in this example.

Given Information



Preliminary Calculations and Assumptions

Identify pertinent properties of the pile selected, ground conditions encountered, and driving tolerances of the pile to be used in the falsework bent.

1. Pile properties ($\emptyset = 12$ in; R = 6 in)

   A $= \pi R^2 = 113$ in^2

S $= \frac{\pi R^3}{4} = 170$ in^3

I $= \frac{\pi R^4}{4} = 1018$ in^4

2. Required Pile Penetration (Section 8-6.04A)

   Minimum $\frac{D}{H} \geq 0.75$; design $\frac{D}{H} = \frac{12}{12} = 1.0$ OK

   Minimum D for construction = (0.75)(12.0) = 9.0 ft

3. Soil Relaxation Factor (Section 8-6.04D)

   Assumptions: (1) normal (average) soil & (2) 30-day time period

   From Soil Factor Chart (Fig. 8-24) R=1.25

4. Point of Pile Fixity (Section 8-6.04B & 8-6.04D)

   Y_1 = kd

   d = 1 ft (pile diam. @ ground line)

   k = 4 (for medium hard to medium soft soil) (8-6.04B-2)

   Y_1 = (4)(1.0) = 4.0 ft

   Y_2 = (Y_1)(soil relax. Factor from fig 8-24) = (4.0)(1.25) = 5.0 ft

5. Driving Tolerances (Section 8-6.04C)

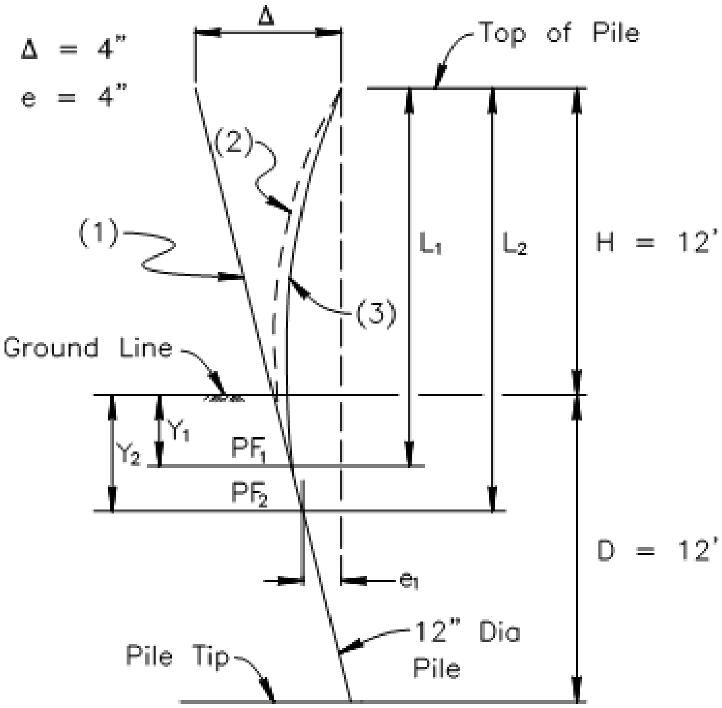
$$\left.\begin{array}{l}\text{Max. pile pull} = \triangle = 4 \text{ in} \\ \text{Max. pile lean} = e_1 = 4 \text{ in}\end{array}\right\} \quad \text{Values from F/W drawings}$$

6. Modulus of Elasticity (NDS Table 6A)

   Assume Pacific Coast Douglas fir: E = 1,700,000 psi

Investigate the Effect of Pile Pull (Section 8-6.05A)

Pile Schematic (no scale)



(1) = Driven position

(2) = Initial pulled position

(3) = Position when P_V is applied

$L_1 = H + Y_1 = 12.0' + 4.0' = 16.0'$

$L_2 = H + Y_2 = 12.0' + 5.0' = 17.0$

1. Calculate F_1 = force to pull pile into line

$$F_1 = \frac{3EI\triangle}{(12L_1)^3} = \frac{3(1.7\text{x}\,10^6)(\,1018)(4)}{(12\text{ x }16.0)^3} = 2934 \text{ lbs}$$

2. Calculate the initial bending stress

$$f_{bp(1)} = \frac{F_1(12L_1)}{S} = \frac{(2934)(16)(12)}{170} = 3314 \text{ psi}$$

3314 psi < 4000 psi allowed (per Section 8-6.05A), therefore OK

3. Calculate F_2 = force after soil relaxes

$$F_2 = \frac{F_1(L_1)^3}{(L_2)^3} = \frac{2934(16.0)^3}{(17.0)^3} = 2446 \text{ lbs}$$

4. Calculate bending stress remaining in pile after soil relaxation (final condition)

$$f_{bp(2)} = \frac{F_2(12L_2)}{S} = \frac{2446(17)(12)}{170} = 2935 \text{ psi}$$

Evaluate System Adequacy (Section 8-6.05E)

1. Determine bent type

   $L_u = Y_2 + (12.0 - 10.0) = 5.0 + 2.0 = 7.0$ ft (the distance from PF_2 to bottom of X-brace)

   $\frac{L_u}{d} = \frac{7.0}{1.0} = 7 < 8$ ∴ Type I bent (Bending stress produced by the horizontal design load may be neglected per Section 8-6.05E(1))

2. Calculate bending stress due to vertical load eccentricity

   Since it's a Type I bent follow Section 8-6.05E(1)

   $f_{be(1)} = \frac{(P_v e_1)}{S} = \frac{(42000)(4)}{170} = 988$ psi

3. Calculate stress due to axial compression

   $f_c = \frac{P_v}{A} = \frac{42000}{113} = 371$ psi

4. Determine allowable compressive stress (Use NDS)

   Note: bent supported at the cap in the longitudinal direction.

   L_u (in longitudinal direction) = L_2 = 17.0 ft (pile is unrestrained in longitudinal direction)

   Equivalent "d" = $r\sqrt{12} = 3\sqrt{12} = 10.39$ in (r = radius of gyration= D/4; NDS C6.3.8)

   $\frac{L_u}{d} = \frac{(17)(12)}{10.39} = 19.63$

   Reference design value in compression F_c = 1300 psi (NDS supplement table 6A)

   Adjustment factors from NDS table 6.3.1:

   $C_D = 1.25$ *Duration Factor for 2% lateral loading NDS 6.3.2*

   $C_t = 1.0$ *Temperature Factor NDS 6.3.4 (Temp up to 100ºF)*

C_{ct} = 1.0 *Condition Treatment factor NDS 6.3.5*
C_P = 0.631 *Column Stability Factor NDS 6.3.8 (Eff length* $17\ ft$*)*
C_{cs} = 1.03 *Critical Section Factor NDS 6.3.9 (tip to point of fixity 7 ft)*
C_{ls} = 1.11 *Load Sharing Factor NDS 6.3.11 (assume continuous cap)*

Adjusted design compression value $F_c' = F_c (C_D)(C_t)(C_{ct})(C_P)(C_{cs})(C_{ls})$ = 1172 psi

5. Solve combined stress expression

$$\frac{f_{bp(2)}+2f_{be(1)}}{3F_b'}+\frac{2f_c}{3F_c'}\le 1.0 \qquad (8\text{-}6.05E(1)\text{-}1)$$

Need to calculate F_b' using NDS

Reference design value F_b = 2050 psi (NDS supplement table 6A)

Adjustment factors from NDS table 6.3.1:

C_D = 1.25 *Duration Factor for 2% lateral loading NDS 6.3.2*
C_t = 1.0 *Temperature Factor NDS 6.3.4 (Temp up to 100ºF)*
C_{ct} = 1.0 *Condition Treatment factor NDS 6.3.5*
C_F = 1.0 *Size Factor NDS 6.3.7*
C_{ls} = 1.08 *Load Sharing Factor NDS 6.3.11 (assume continuous cap)*

Adjusted design compression value $F_b' = F_b (C_D)(C_t)(C_{ct})(C_F)(C_{ls})$ = 2768 psi

Substitute values and solve combined stress equation

$$\frac{2935+2(988)}{3(2768)}+\frac{2(371)}{3(1172)}$$

$0.59 + 0.21 = 0.80 \le 1.0$ System is adequate!!

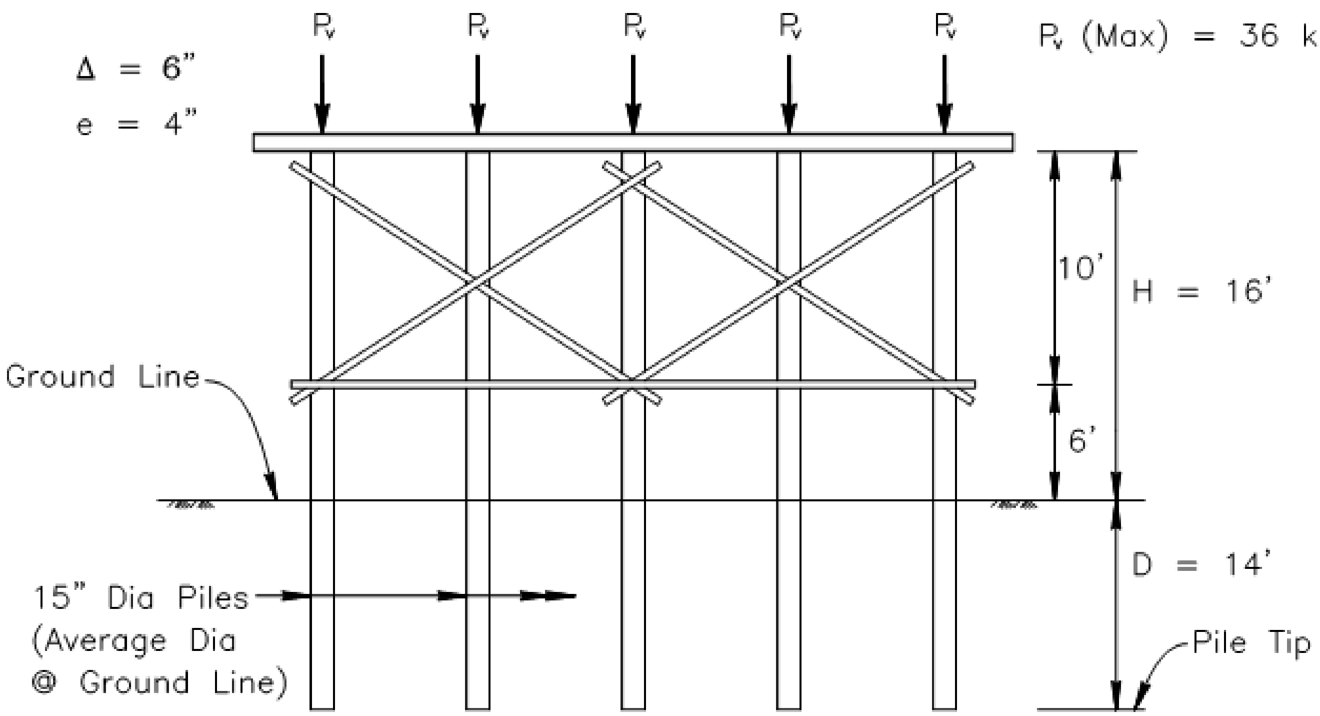
Options available if combined stress > 1:

a. Use larger diameter pile
b. Reduce allowable values for $\triangle$ and/or e_1
c. Shorten F/W span to reduce P_v

Appendix D Example 25 – Timber Pile Bents – Type II Bent

Refer to *Falsework Manual,* Section 8-6.05, *Analysis of Timber Pile Bents.* Occasionally pile foundations will be used for falsework systems due to poor soil conditions, having to traverse over water, and to mitigate differential settlement. As-built conditions of the driven piles will dictate the bent capacity to resist horizontal loads. Type II falsework bents are analyzed in this example.

Given Information



Preliminary Calculations and Assumptions

1. Pile properties (15"∅ pile; R = 7.5″)

$$A = \pi R^2 = 177 \text{ in}^2$$

$$S = \frac{\pi R^3}{4} = 331 \text{ in}^3$$

I $= \frac{\pi R^4}{4} = 2485 \text{ in}^4$

2. Required pile penetration (Section 8-6.04A)

   Minimum $\frac{D}{H} = 0.75$; design $\frac{D}{H} = \frac{14}{16} = 0.875$ OK

   Minimum D for construction = (0.75)(16.0)= 12.0'

3. Soil relaxation factor (Section 8-6.04D)

   Assumptions: (1) normal (average) soil & (2) 30-day time period

   From Soil Factor Chart (Figure 8-12) R = 1.25

4. Point of pile fixity (Section 8-6.04B 8-6.04D):

   Y_1 = (4)(pile diameter @ ground line) = (4)(1.25) = 5.0'

   Y_2 = (Y_1)(soil relax. factor) = (5.0)(1.25) = 6.25'

5. Driving tolerances (Section 8-6.04C)

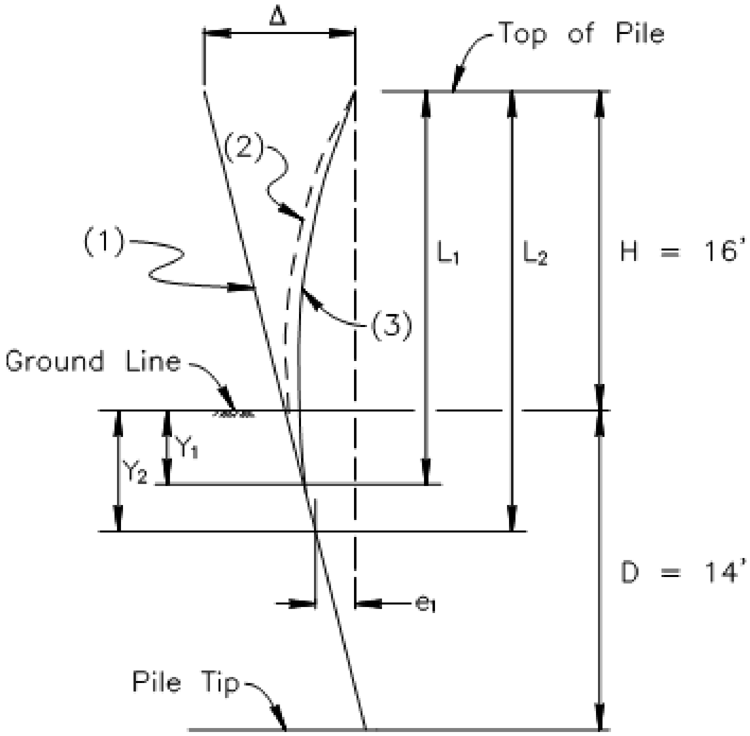
   $\left.\begin{array}{l}\text{Max. pile pull} = \triangle = 6''\\ \text{Max. pile lean} = e_1 = 4''\end{array}\right\}$ Values from F/W drawings

6. Modulus of Elasticity (NDS Table 6A)

   Assume Pacific Coast Douglas fir: E = 1,700,000 psi

Investigate the Effect of Pile Pull (Section 8-6.05A)

Pile Schematic (no scale)



(1) = Driven position

(2) = Initial pulled position

(3) = Position when P_V is applied

Y_1 = 5.0' ; Y_2 = 6.25'

$L_1 = H + Y_1$ = 21.0'

$L_2 = H + Y_2$ = 22.25'

1. Calculate force to pull pile into line

$$F_1 = \frac{3EI\triangle}{(12L_1)^3} = \frac{3(1.7 \text{ x } 10^6)(2485)(6)}{(12 \text{ x } 21.0)^3} = 4752 \text{ lbs}$$

2. Calculate the initial bending stress

$$f_{bp(1)} = \frac{F_1(12L_1)}{S} = \frac{(4752)(12 \text{ x } 21.0)}{331} = 3618 \text{ psi}$$

3618 psi < 4000 psi allowed. OK

3. Calculate force remaining when P_v is applied

$$F_2 = \frac{F_1(L_1)^3}{(L_2)^3} = \frac{4752(21.0)^3}{(22.25)^3} = 3995 \text{ lbs}$$

4. Calculate relaxed bending stress

$$f_{bp(2)} = \frac{F_2(12L_2)}{S} = \frac{3995(12 \text{ x } 22.25)}{331} = 3223 \text{ psi}$$

Evaluate System Adequacy (Section 8-6.05E)

1. Determine bent type

   $L_u = Y_2 + (16.0 - 10.0) = 6.25 + 6.0 = 12.25$ ft

   $\frac{L_u}{d} = \frac{12.25}{1.25} = 9.8$; ∴ Type II bent

   Consider H but not P-delta - See Section 8-6.05F(2)

2. Calculate stress due to pile lean or load eccentricity

   $$f_{be(1)} = \frac{P_v(e_1)}{S} = \frac{36000(4)}{331} = 435 \text{ psi}$$

3. Calculate stress due to design H

   H = (0.02)(36000) = 720 lbs - See Standard Specifications Section 48-2.02B(2)

   $$f_{bH} = \frac{(H)(L_u)}{S} = \frac{(720)(12.25 \text{ x } 12)}{331} = 320 \text{ psi}$$

4. Calculate horizontal displacement

   $$X = \frac{H(12L_u)^3}{3EI} = \frac{720(12 \text{ x } 12.25)^3}{3(1.7 \text{ x } 10^6)(2485)} = 0.18 \text{ in} = e_2$$

5. Calculate stress due to additional P_v eccentricity

   $$f_{be(2)} = \frac{P_v(e_2)}{S} = \frac{36000(0.18)}{331} = 19.6 \text{ psi}$$

6. Calculate stress due to axial compression

   $$f_c = \frac{P_v}{A} = \frac{36000}{177} = 203 \text{ psi}$$

7. Determine allowable compressive stress (Use NDS)

   Note: bent supported at the cap in the longitudinal direction.

L_u (in longitudinal direction) = L_2 = 22.25 ft (pile is unrestrained in longitudinal direction)

Equivalent “d” = $\mathrm{r}\sqrt{12} = 3.75\sqrt{12} =$ 13 in (r = radius of gyration= D/4; NDS C6.3.8)

$$\frac{L_u}{d} = \frac{(22.25)(12)}{13} = 20.54$$

Reference design value in compression F_c = 1300 psi (NDS supplement table 6A)

Adjustment factors from NDS table 6.3.1:

C_D = 1.25 *Duration Factor for 2% lateral loading NDS 6.3.2*
C_t = 1.0 *Temperature Factor NDS 6.3.4 (Temp up to 100ºF)*
C_{ct} = 1.0 *Condition Treatment factor NDS 6.3.5*
C_P = 0.593 *Column Stability Factor NDS 6.3.8 (Eff length* $22.25\ ft$*)*
C_{cs} = 1.03 *Critical Section Factor NDS 6.3.9 (tip to point of fixity 7.75 ft)*
C_{ls} = 1.11 *Load Sharing Factor NDS 6.3.11 (assume continuous cap)*

Adjusted design compression value F_c' = F_c $(C_D)(C_t)(C_{ct})(C_P)(C_{cs})(C_{ls})$ = **1102 psi**

8. Determine allowable bending stress (Use NDS)

Reference design value in compression F_b = 2050 psi (NDS supplement table 6A)

Adjustment factors from NDS table 6.3.1:

C_D = 1.25 *Duration Factor for 2% lateral loading NDS 6.3.2*
C_t = 1.0 *Temperature Factor NDS 6.3.4 (Temp up to 100ºF)*
C_{ct} = 1.0 *Condition Treatment factor NDS 6.3.5*
C_F = 0.99 *Size Factor NDS 6.3.7*
C_{ls} = 1.08 *Load Sharing Factor NDS 6.3.11 (assume continuous cap & note that this value is different depending on Compression or Bending!)*

Adjusted design bending value F_b' = F_c $(C_D)(C_t)(C_{ct})(C_F)(C_{ls})$ = **2740 psi**

9. Check pile adequacy using combined stress expression

$$\frac{f_{bp(2)} + 2f_{be(1)} + 2\,[f_{bH} + f_{be(2)}]}{3F'_b} + \frac{2f_c}{3F'_c} \ngtr 1.0$$

$$\frac{3223 + 2(435) + 2(320 + 19.6)}{3(2740)} + \frac{2(203)}{3(1102)}$$

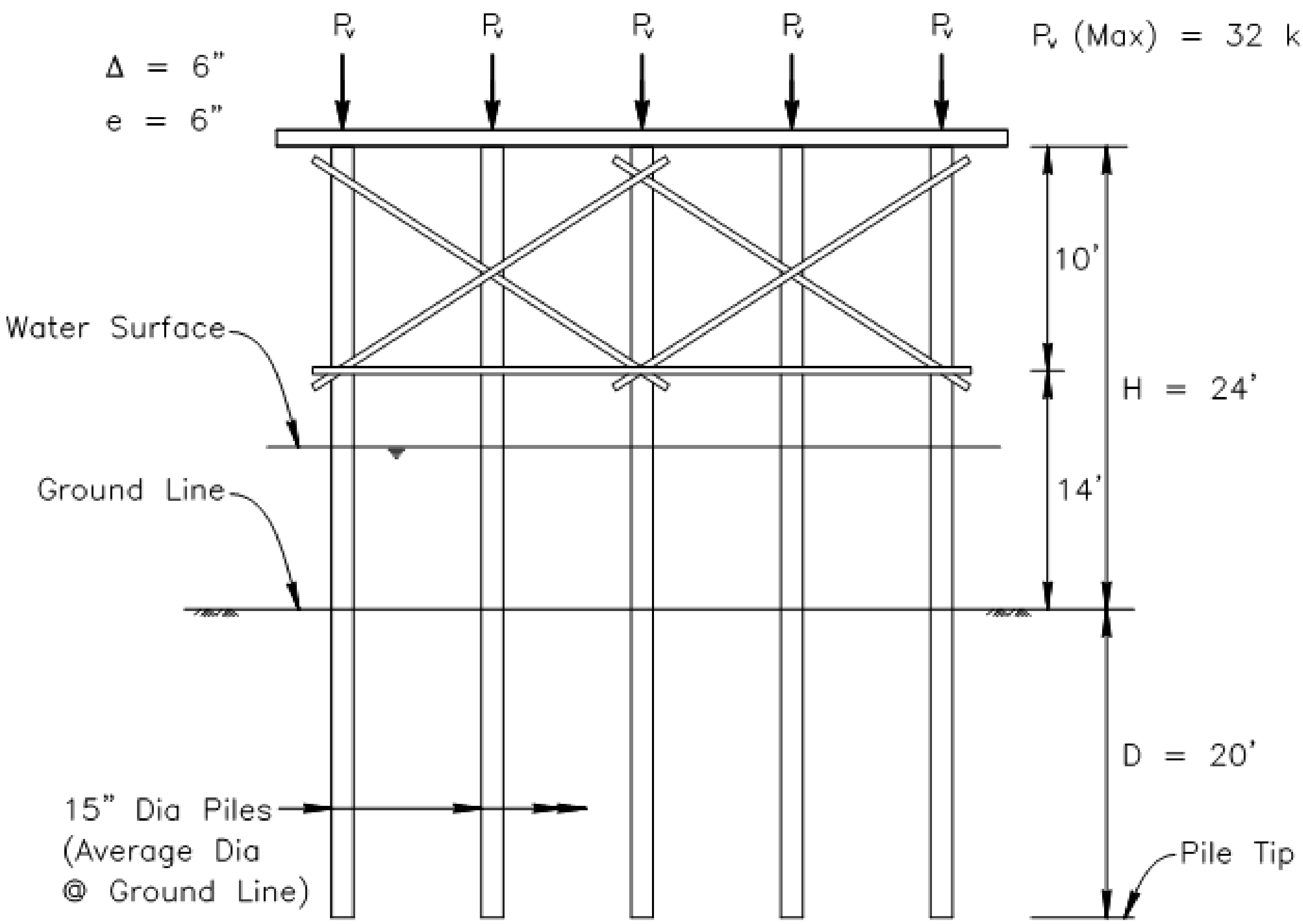
0.58 + 0.12 = 0.70 < 1.0

<u>System is adequate!</u>

Appendix D Example 26 – Timber Pile Bents – Type III Bent

Refer to *Falsework Manual,* Section 8-6.05, *Analysis of Timber Pile Bents.* Occasionally pile foundations will be used for falsework systems due to poor soil conditions, having to traverse over water, and to mitigate differential settlement. As-built conditions of the driven piles will dictate the bent capacity to resist horizontal loads. Type III falsework bents are analyzed in this example.

Given Information



Preliminary Calculations and Assumptions

1. Pile properties (15in∅ pile; R = 7.5 in)

   A $= \pi R^2 = 177$ in^2

   S $= \frac{\pi R^3}{4} = 331$ in^3

   I $= \frac{\pi R^4}{4} = 2485$ in^4

2. Required pile penetration (Section 8-6.04A)

   Minimum $\frac{D}{H} = 0.75$; design $\frac{D}{H} = \frac{20}{24} = 0.83$ OK

   Minimum D for construction = (0.75)(24)= 18.0 ft

3. Soil relaxation factor (Section 8-6.04D)

   Assumptions: (1) normal (average) soil & (2) 30-day time period

   From Soil Factor Chart (Figure 8-24) R = 1.25

4. Point of pile fixity (Section 8-6.04B & Section 8-6.04D)

   Y_1 = (4)(ground line pile diameter) = (4)(1.25) = 5.0 ft

   Y_2 = (Y_1)(soil relax. factor) = (5.0)(1.25) = 6.25 ft

5. Driving tolerances (Section 8-6.04C)

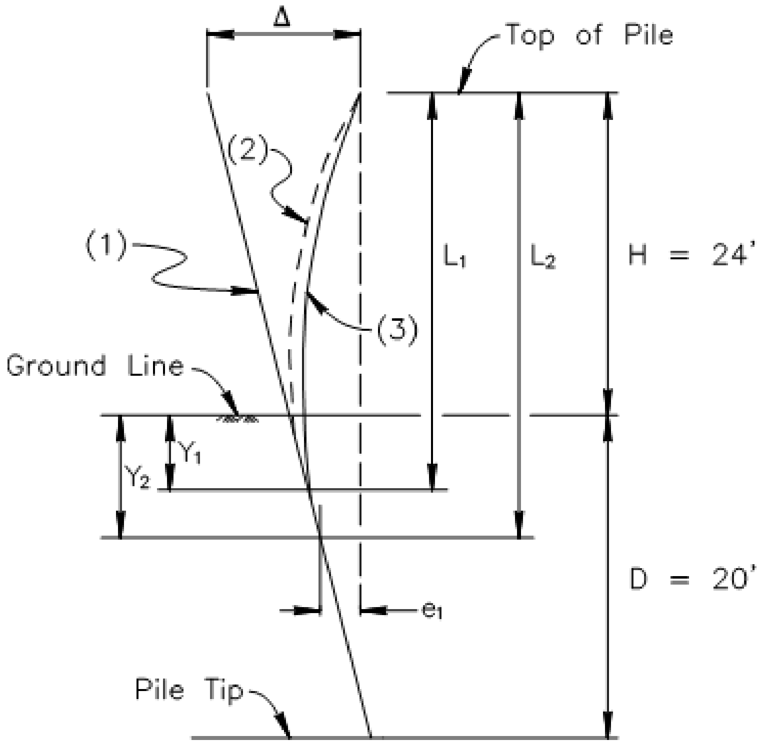
   $\left.\begin{array}{l}\text{Max. pile pull} = \triangle = 6\text{in} \\ \text{Max. pile lean} = e_1 = 6\text{in}\end{array}\right\}$ Values from F/W drawings

6. Modulus of Elasticity (NDS Table 6A):

   Assume Pacific Coast Doulas Fir: E = 1,700,000 psi

Investigate the Effect of Pile Pull

Pile Schematic (no scale)



(1) = *Driven position*

(2) = *Initial pulled position*

(3) = *Position when* P_V *is applied*

$Y_1 = 5.0'$; $Y_2 = 6.25\text{ft}$

$L_1 = H + Y_1 = 29.0\text{ft}$

$L_2 = H + Y_2 = 30.25\text{ft}$

1. Calculate force to pull pile into line (Section 8-6.05A)

$$F_1 = \frac{3EI\triangle}{(12L_1)^3} = \frac{3(1.7 \text{ x } 10^6)(2485)(6)}{(12 \text{ x } 29.0)^3} = 1804 \text{ lbs}$$

2. Calculate the initial bending stress

$$f_{bp(1)} = \frac{F_1(12L_1)}{S} = \frac{(1804)(12 \text{ x } 29.0)}{331} = 1897 \text{ psi}$$

1897 < 4000 psi allowed. <u>OK</u>

3. Calculate force remaining when P_v is applied

$$F_2 = \frac{F_1(L_1)^3}{(L_2)^3} = \frac{1804(29.0)^3}{(30.25)^3} = 1589 \text{ lbs}$$

4. Calculate the relaxed bending stress

$$f_{bp(2)} = \frac{F_2(12L_2)}{S} = \frac{(1589)(12 \text{ x } 30.25)}{331} = 1743 \text{ psi}$$

Evaluate System Adequacy (Section 8-6.05E)

1. Determine bent type

   $L_u = Y_2 + (24.0 – 10.0) = 6.25 + 14.0 = 20.25$ ft

   $$\frac{L_u}{d} = \frac{20.25}{1.25} = 16.2 > 15, \quad \therefore \text{ Type III bent}$$

   Consider P-delta effect – See Section 8-6.05E(3)

2. Calculate stress due to pile lean

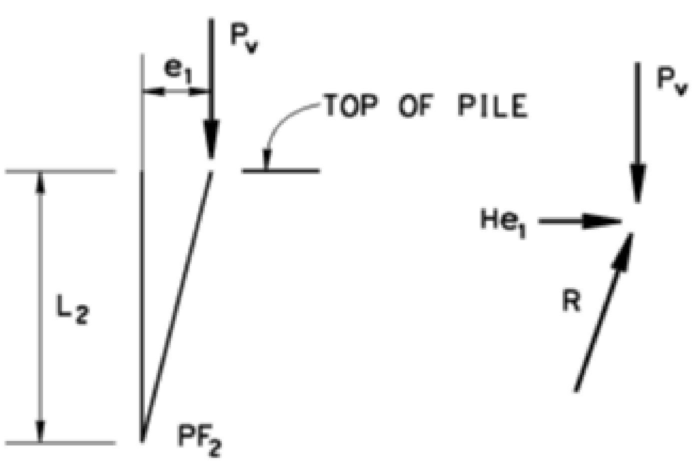
   $$f_{be(1)} = \frac{P_v(e_1)}{S} = \frac{(32000)(6)}{331} = 580 \text{ psi}$$

3. Calculate stress due to design H (2% of gravity load for lateral load)

   H = (0.02)(32,000) = 640 lbs

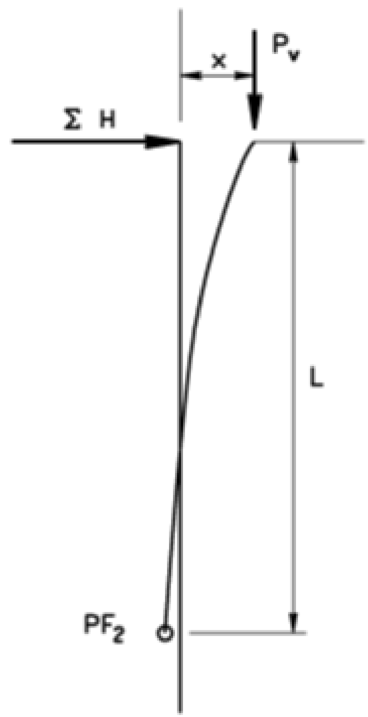
   $$f_{bH} = \frac{(H)(L_u)}{S} = \frac{(640)(12*20.25)}{331} = 470 \text{ psi}$$

4. Calculate horizontal component of P_v reaction



$$He_1 = \frac{P_v\,(e_1)}{12\ L_2} = \frac{(32000)(6)}{12(30.25)} = 529 \text{ lbs}$$

5. Calculate total horizontal displacement (e_3):



X = displacement due to total H load

$$X = \frac{\sum H\,(L)^3}{3EI}$$

$$\sum H = \text{design H} + He_1$$

= 640 + 529 = 1169 lbs

$12L_u$ = 12(20.25 ft = 243 in

3EI = 3(1.7x10^6 x 2485) =1.267x10^{10} lb-in^2

Refer to Section 8-6.05E(3)

X = $\frac{(1169)(243)^3}{3EI}$ =1.32	H_1 = 1169 + $\frac{(32000)(1.32)}{243}$ =1343 lbs
X_1 = $\frac{(1343)(243)^3}{3EI}$ =1.52	H_2 = 1343 + $\frac{(32000)(1.52-1.32)}{243}$ =1369 lbs
X_2 = $\frac{(1369)(243)^3}{3EI}$ =1.55	Values within 5% STOP

6. Calculate bending stress due to ΣH displacement

$$f_{be3} = \frac{P_v(e_3)}{S} = \frac{(32000)(1.55)}{331} = 150 \text{ psi}$$

7. Calculate stress due to axial compression

$$f_c = \frac{P_v}{A} = \frac{32000}{177} = 181 \text{ psi}$$

8. Determine allowable compressive stress (Use NDS)

Note: Bent is supported at the cap in the longitudinal direction.

L_u (in longitudinal direction) = L_2 = 30.25'

Equivalent "d" = $r\sqrt{12} = 3.75\sqrt{12} =$ 13 in (r = radius of gyration= D/4; NDS C6.3.8)

$$\frac{L_u}{d} = \frac{12 \text{ x } 30.25}{13} = 27.9$$

Reference design value in compression F_c = 1300 psi (NDS supplement table 6A)

Adjustment factors from NDS table 6.3.1:

C_D = 1.25 *Duration Factor for 2% lateral loading NDS 6.3.2*
C_t = 1.0 *Temperature Factor NDS 6.3.4 (Temp up to 100ºF)*
C_{ct} = 1.0 *Condition Treatment factor NDS 6.3.5*
C_P = 0.351 C*olumn Stability Factor NDS 6.3.8 (Eff length* $30.25f$*)*
C_{cs} = 1.06 *Critical Section Factor NDS 6.3.9 (tip to point of fixity 13.75 ft)*
C_{ls} = 1.11 *Load Sharing Factor NDS 6.3.11 (assume continuous cap)*

Adjusted design compression value F_c' = F_c (C_D)(C_t)(C_{ct}) (C_P)(C_{cs})(C_{ls}) = **671 psi**

9. Determine allowable bending stress (Use NDS)

Reference design value in compression F_b = 2050 psi (NDS supplement table 6A)
Adjustment factors from NDS table 6.3.1:

C_D = 1.25 *Duration Factor for 2% lateral loading NDS 6.3.2*
C_t = 1.0 *Temperature Factor NDS 6.3.4 (Temp up to 100ºF)*
C_{ct} = 1.0 *Condition Treatment factor NDS 6.3.5*
C_F = 0.99 *Size Factor NDS 6.3.7*
C_{ls} = 1.08 *Load Sharing Factor NDS 6.3.11 (assume continuous cap & note that this value is different depending on Compression or Bending!)*
Adjusted design bending value F_b' = F_c (C_D)(C_t)(C_{ct}) (C_F) (C_{ls}) = **2740 psi**

10. Check pile adequacy using combined stress expression

$$\frac{f_{bp(2)}+2f_{be(1)}+2(f_{bH}+f_{be(3)})}{3F'_b} + \frac{2f_c}{3F'_c} \not< 1.0$$

$$\frac{1743+2(580)+2(470+150)}{3(2740)}+\frac{2(181)}{3(671)}$$

0.50+ 0.18= 0.68 < 1.0

System is adequate!

Appendix D Example 27 – Pile Penetration Failure – Type I Bent

Refer to *Falsework Manual,* Section 8-6.06A, *Failure to Attain Required Penetration.* When the D/H ratio is less than 0.75 but greater than or equal to 0.45 for pile foundations an alternative procedure is used for analysis of piles (Ref Section 8-6.06A). This condition will increase rotation of the falsework bent which will reduce bending resistance and overall load-carrying capacity. This example demonstrates the alternate procedure for pile analysis.

Given Information

Refer the example in Section D-24, *Type I Bent,* and assume the pile failed to reach its required minimum depth (D) of 9-feet (Refer to Section 8-6.04A).

Revised design using the same 12″ piles but penetrating only 7.5-feet. Assume the same pile pull or pile lean values.

Determine Adequacy of Pile

1. Find new value for L_2 - See Section 8-6.06A(1)

   New $\frac{D}{H} = \frac{7.5}{12} = 0.625$

   From Figure 8-29, Q = 1.10 (for normal soil)

   Y_2 = 4dR = $4\left(\frac{12}{12}\right)(1.25) = 5.00$ ft (see example 24)

   New $L_2 = H + (Q)(Y_2) = 12 + (1.10)(5.00) = 17.50$ ft

2. Recalculate $f_{bp(2)}$ using new L_2

   $$F_2 = \frac{3EI\triangle}{(12L_2)^3} = \frac{3(1.7 \text{ x } 10^6)(1018)(4)}{\{(12)(17.50)\}^3} = 2242 \text{ lb}$$

   $$f_{bp(2)} = \frac{F_2(12L_2)}{S} = \frac{(2242)(12)(17.50)}{170} = 2770 \text{ psi}$$

3. Check bent type

   New L_u = 2.0 + new Y_2 = 2.0 + (1.10)(5.00) = 7.5 ft

   $\frac{L_u}{d} = \frac{7.50 \text{ x } 12}{12} = 7.50 < 8.0$ <u>Type I bent</u>

4. Evaluate system adequacy

   $f_{be(1)}$ and f_c are unchanged (see Example 24)

   L_u (in longitudinal direction governs) = L_2 = 17.50 ft

   Equivalent d = 10.39 in (see Example 24)

   Capacity in compression:

   Reference design value in compression F_c = 1300 psi (NDS supplement table 6A)

   Adjustment factors from NDS table 6.3.1:

C_D = 1.25	*Duration Factor for 2% lateral loading*
C_M = 1.0	*Wet Service Factor NDS 6.3.3*
C_t = 1.0	*Temperature Factor NDS 6.3.4 (Temp up to 100ºF)*
C_{ct} = 1.0	*Conditioning Treatment Factor NDS 6.3.5 (air dried)*
C_{cs} = 1.01	*Critical Section Factor NDS 6.3.9 (L_c = 2.50 ft)*
C_P = 0.615	*Column Stability Factor NDS 3.7.1 (Eff length $17.50\ ft$)*
C_{ls} = 1.11	*Load sharing Factor NDS 6.3.11*

   Adjusted design compression value $F_c' = F_c (C_D)(C_M)(C_t)(C_{ct})(C_{cs})(C_P)(C_{ls})$ = 1120 psi

<u>Solve combined stress expression (Type I)</u>

F_b' = 2768 psi (See example problem D-24)

$$\frac{f_{bp(2)}+2f_{be(1)}}{3F'_b} + \frac{2f_c}{3F'_c} \leq 1.0$$

$$\frac{2770+2(988)}{3(2768)} + \frac{2(372)}{3(1120)} = 0.57 + 0.22 = 0.79 \leq 1.0 \ \underline{\text{OK}}$$

Appendix D Example 28 – Pile Penetration Failure – Type II Bent

Refer to *Falsework Manual,* Section -8-6.06A, *Failure to Attain Required Penetration.* When the D/H ratio is less than 0.75 but greater than or equal to 0.45 for pile foundations an alternative procedure is used for analysis of piles (Reference Section 8-6.06A). This condition will increase rotation of the falsework bent which will reduce bending resistance and overall load-carrying capacity. This example demonstrates the alternate procedure for pile analysis.

Given Information

Refer the example in Section D-25, *Type II Pile,* and assume the critical pile in this example has the following as-driven values:

	Planned	Actual
D	14 ft (min)	10 ft
$\triangle$	6 in (max.)	6 in
e_1	4 in (max.)	8 in at 60° angle (relative brg.) with $\triangle$

Determine Adequacy of Pile (See Section 8-6.05E(2))

1. Check adequacy of pile penetration

   $$\frac{\mathrm{D}}{\mathrm{H}} = \frac{10}{16} = 0.625$$

   $0.45 \leq 0.625 \leq 0.75$ Determine stiffness reducing coefficient (Q)

   From Figure 8-29, Q = 1.10 (for normal soil)

2. Find new values for Y_2 and L_2

   $Y_2 = Q(Y_2) = (1.10)(6.25) = 6.88$ ft

   $L_2 = H + Y_2 = 16.0 + 6.88 = 22.88$ ft

3. Check bent type

L_u = Dist. FG to Brace + Y_2 = 6 + 6.88 = 12.88 ft

$$\frac{L_u}{d} = \frac{(12.88)(12)}{15} = 10.30 \qquad 8 < 10.3 \leq 15 \text{ (Eq 8-6.05E-2) } \underline{\text{Still Type II bent}}$$

4. Calculate stress due to pile pull

$$F_2 = \frac{3EI\triangle}{(12L_2)^3} = \frac{3(1.7 \text{ x } 10^6)(2485)(6)}{(12 \text{ x } 22.88)^3} = 3674 \text{ lbs}$$

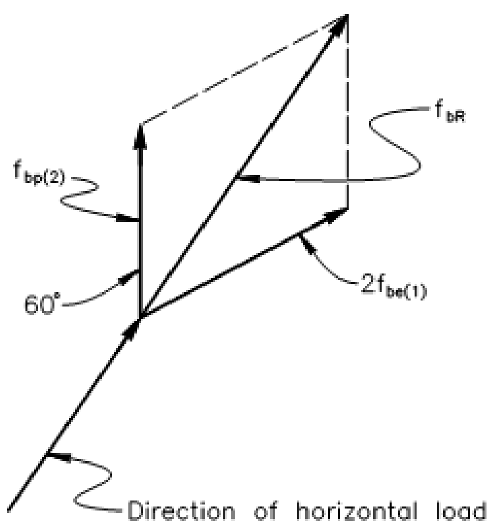
$$f_{bp\,(2)} = \frac{F_2(12L_2)}{S} = \frac{(3674)(12 \text{ x } 22.88)}{331} = 3048 \text{ psi}$$

Note that it is not necessary to calculate the initial bending stress for this pile because △ is unchanged. (The longer L_1 length will give a corresponding lower value for $f_{bp(1)}$).

5. Calculate stress due to pile lean (see example D-25 for vertical load)

$$f_{be(1)} = \frac{P_v(e_1)}{S} = \frac{(36000)(8)}{331} = 870 \text{ psi}$$

6. Calculate stress resultant – See Section 8-6.06C



$f_{bp(2)}$ = 3048 psi

2 $f_{be(1)}$ = 1740 psi (load factor = 2)

Solve stress vector triangles to find the resultant stress

$$f_{bR} = \sqrt{3048^2 + 1740^2 + 2(3048)(1740)\text{Cos } 60}$$

f_{bR} = 4198 psi

7. Calculate stress due to design horizontal load (H)

H = 720 lbs (2%(DL) from example D-25)

L_u = 12.88 ft (See step 3)

$$f_{bH} = \frac{H(12L_u)}{S} = \frac{(720)(12)(12.88)}{331} = 336 \text{ psi}$$

8. Calculate horizontal displacement

$$X = \frac{H(12L_u)^3}{3EI} = \frac{(720)(12 \text{ x } 12.88)^3}{3(1.7 \text{ x } 10^6)(2485)} = 0.21 \text{ in} = e_2$$

9. Calculate stress due to e_2

$$f_{be(2)} = \frac{P_v(e_2)}{S} = \frac{(36000)(0.21)}{331} = 22.8 \text{ psi}$$

10. Determine allowable compressive stress

Note: actual f_c is unchanged at 203 psi (see step 6 example D-25)

$L_u = L_2$ = 22.88 ft (long. direction governs)

Capacity in compression:

Reference design value in compression F_c = 1300 psi (NDS supplement table 6A)

Adjustment factors from NDS table 6.3.1:

CD = 1.25	Duration Factor for 2% lateral loading
CM = 1.0	Wet Service Factor NDS 6.3.3
Ct = 1.0	Temperature Factor NDS 6.3.4 (Temp up to 100ºF)
Cct = 1.0	Conditioning Treatment Factor NDS 6.3.5 (air dried)
Ccs = 1.01	Critical Section Factor NDS 6.3.9 (Lc = 3.12 ft)
CP = 0.577	Column Stability Factor NDS 3.7.1 (Eff length 22.88 ft)
Cls = 1.11	Load sharing Factor NDS 6.3.11

Adjusted design compression value $F_c' = F_c (C_D)(C_M)(C_t)(C_{ct})(C_{cs}) (C_P)(C_{ls})$ = 1051 psi

Capacity in compression:

Reference design value Fb = 2050 psi (NDS supplement table 6A)

Adjustment factors from NDS table 6.3.1:

$C_D = 1.25$	*Duration Factor for 2% lateral loading NDS 6.3.2*
$C_t = 1.0$	*Temperature Factor NDS 6.3.4 (Temp up to 100ºF)*
$C_{ct} = 1.0$	*Condition Treatment factor NDS 6.3.5*
$C_F = 0.99$	*Size Factor NDS 6.3.7*
$C_{ls} = 1.08$	*Load Sharing Factor NDS 6.3.11 (analyze individual pile capacity)*

Adjusted design compression value $F_{b'} = F_b\,(C_D)(C_t)(C_{ct})\,(C_F)(C_{ls}) = 2740$ psi

Solve combined stress equation

$$\frac{f_{bR} + 2(f_{bH} + f_{be\,(2)})}{3F'_b} + \frac{2f_c}{3F'_c} \leq 1.0$$

$$\frac{4198 + 2(336 + 22.8)}{3(2740)} + \frac{2(203)}{3(1051)} \leq 1.0$$

0.60 + 0.13 = 0.73 ≤ 1.0 **<u>OK</u>**

Appendix D Example 29 – Short Poured-In-Place Concrete Piles

The following section presents sample calculations for specific items discussed in the subsections of Section 5-6, *Short Poured-In-Place Concrete Piles*. For a full example problem see Appendix D, Example 30 – *Short Poured-In-Place Concrete Piles.*

Pile Uplift in Cohesionless Soil:

Refer to Section 5-6.02A, *Pile Uplift in Cohesionless Soil*.

Given Information

Soil internal friction:
angle ϕ = 30°

Unit weight of concrete:
$\gamma_c = 145$ pcf

Unit weight of soil:
γ_s = 100 pcf

Water table:
4 feet from pile tip at anticipated time of load application

Pile:
L_p = Length of the pile = 12 ft
d = Pile diameter = 18 in = 1.5 ft
z = Depth below ground = 10 ft

Single use loading (FS = 2)

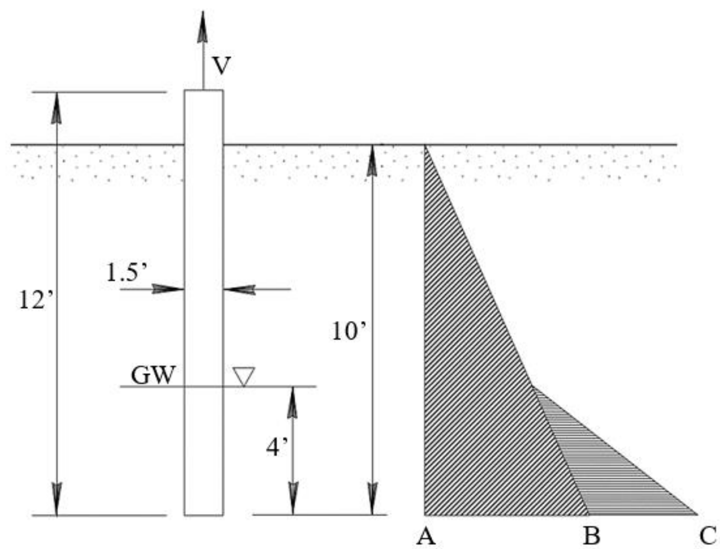


Figure D-29-1. Short Concrete Pile in Cohesionless Soil

Determine vertical load capacity for the poured-in-place concrete pile in Cohesionless Soil

$$R = \pi dzS + W \quad (5\text{-}6.02\text{-}1)$$

$$S = \beta\sigma_z \leq 4{,}000 \text{ psf} \quad (5\text{-}6.02A\text{-}1)$$

$$\beta = 1.5 - 0.315\, z^{1/2} \text{ but } 0.25 \leq \beta \leq 1.2 \quad (5\text{-}6.02A\text{-}2)$$

Where:

- R = Resistance to pile uplift (lb)
- S = Unit shearing resistance on the soil-pile interface (psf)
- W = Pile weight (lbs)
- β = Reduction factor for cohesionless soils
- σ_z = Effective overburden soil weight (psf). Below the water table the weight of water is subtracted from the soil unit weight so that only the submerged soil weight is used
- AB = The pressure due to the weight of the soil
- BC = The pressure due to the weight of the water

Unit shearing resistance

$\beta = 1.5 - 0.315\, z^{1/2} = 1.5 - 0.315(10)^{1/2} = 0.5 \quad 0.25 \leq \beta \leq 1.2$ **OK**

$z = 10$ ft (z_{dry} = 6 ft; $z_{submerged}$ = 4 ft)

$\sigma_z = 6(100) + 4(100 - 62.4) = 750$ pcf

$S = \beta\sigma_z = 0.5(750) = 375 \text{ psf} < 4000 \text{ psf}$ **OK**

Net pile shearing resistance

R_s = (Pile surface area) S = $\pi dzS = \pi(1.5)(10)(375)$ = 17,671 lbs

Pile weight

$W = \pi\left(\frac{d}{2}\right)^2 L_p\, \gamma_c = \pi\left(\frac{1.5}{2}\right)^2 (12)(145)$ = 3075 lbs

Resistance to pile uplift

R = Net pile shearing resistance (R_s) + Pile weight (W) = $\pi dzS + W$

= 17,671 + 3,075 = 20,746 lbs

Working load

$$(V) = \frac{\text{Ultimate Load}}{\text{FS}} = \frac{(20{,}746)}{2} = 10{,}373 \text{ lbs}$$

Pile Uplift in Cohesive Soil:

Refer to Section 5-6.02B, *Pile Uplift in Cohesive Soil*.

Given Information

Unit weight of concrete:
$\gamma_c = 145$ pcf

Soil cohesion:
C = undrained shear strength = 910 psf

Unit weight of soil:
γ_s = 110 pcf

Pile:
L_p = Length of the pile = 12 ft
d = Pile diameter = 18 in = 1.5 ft
z = Depth below ground = 10 ft

Single use loading (FS = 2)

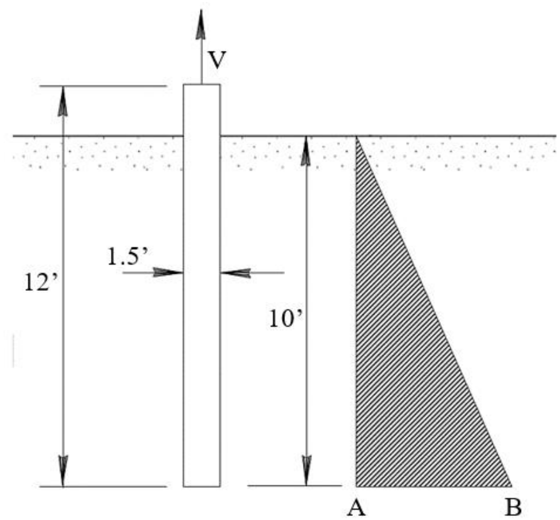


Figure D-29-2. Short Concrete Pile in Cohesive Soil

Determine vertical load capacity for the poured-in-place concrete pile in Cohesive Soil

$$R_s = \pi dzS \tag{5-6.02B-1}$$

Where:

R_s = Shearing resistance (lbs)
S = Unit shearing resistance (psf)
a_z = An empirical unitless reduction factor which accounts for clay shrinkage and lateral pile loadings
AB = The pressure due to the weight of the soil

Unit shearing resistance

$$S = a_z C \leq 5{,}500 \text{ psf} \tag{5-6.02B-2}$$

Use reduction factor for pile diameter d ≤ 18", pile length with more than 5 feet embedment

For $0 \leq z \leq 5$ feet

$$a_{z(0-5)} = (0.055)z \qquad (5\text{-}6.02B\text{-}6)$$

$$= (0.055)5 = 0.275$$

$$S_{(0-5)} = a_{z(0-5)}C = 0.275(910) = 250 \text{ psf} \leq 5500 \text{ psf}$$ **OK**

For z > 5 feet:

$$a_{z(>5)} = 0.55 \qquad (5\text{-}6.02B\text{-}8)$$

$$S_{(>5)} = a_{z(>5)}\,C = 0.55(910) = 500 \text{ psf} \leq 5{,}500 \text{ psf}$$ **OK**

Net pile shearing resistance

$$R_s = \pi d[(5)S_{(0-5)} + (z-5)\,S_{(>5)}]$$

$$= \pi\,(1.5)[(5)(250) + (10-5)(500)]$$

$$= \pi\,(1.5)(3750) = 17{,}671 \text{ lbs}$$

Pile weight

$$W = \pi\left(\frac{d}{2}\right)^2 L_p\,\gamma_c = \pi\left(\frac{1.5}{2}\right)^2 (12)(145) = 3075 \text{ lbs}$$

Resistance to pile uplift

R = Net pile shearing resistance (R_s) + Pile weight (W)

= 17,671 + 3,075 = 20,746 Lbs

Working load

$$V = \frac{\text{Ultimate Load}}{\text{FS}} = \frac{20{,}746}{2} = 10{,}373 \text{ lbs}$$

Lateral Loading in Cohesionless Soil:

Refer to Section 5-6.03A, *Lateral Loading in Cohesionless Soil*.

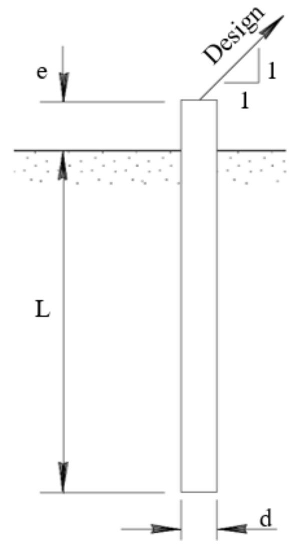


Figure D-29-3. Pile Lateral Loading in Cohesionless Soil

Given Information

Soil internal friction:
angle ϕ = 30°

Unit weight of concrete:
$\gamma_c = 145$ pcf

Unit weight of soil:
γ_s = 110 pcf

Pile:
L = Depth below ground = 8 ft
e = Length from ground surface to ultimate lateral load = 2 ft
d = Pile diameter = 18 in = 1.5 ft

Single use loading (FS = 2)

Determine allowable loading for the poured-in-place concrete pile in Cohesionless Soil

Working load value for lateral load H

$K_p = \tan^2\left(45° + \frac{\phi}{2}\right) = \tan^2\left(45° + \frac{30°}{2}\right)$ = 3.00 (for level ground surface)

L/d = 8/1.5 = 5.33 e/d = 2/1.5 = 1.33

Use Figure 5-23 to find $\frac{H_{ULT}}{K_p \gamma_s d^3}$

$\frac{H_{ULT}}{K_p \gamma_s d^3} \approx 5$ when e = 2'- 0"

$H_{ULT} = 5 \times K_p \gamma_s d^3 = 5 \times (3.0)(110)(1.5)^3$ = 5569 lbs

Working Load Value for H = $\frac{H_{ULT}}{FS} = \frac{5569}{2}$ = 2784 lbs

Working load value for moment M

$$(f_g)^2 = \frac{H_{ULT}}{1.5\ \gamma_s\ dK_p} \qquad (5\text{-}6.03A\text{-}1)$$

$$f_g = \left(\frac{H_{ULT}}{1.5\ \gamma_s\ dK_p}\right)^{\frac{1}{2}} = \left(\frac{5569}{1.5\ (110)(1.5)(3.0)}\right)^{\frac{1}{2}} = 2.74 \text{ ft}$$

$$M_{ULT} = H_{ULT}\left(e + \frac{2f_g}{3}\right) \qquad (5\text{-}6.03A\text{-}2)$$

$$= 5569\left(2 + \frac{(2)(2.74)}{3}\right) = 21{,}311 \text{ ft-lb}$$

Working Load Value for M = $\frac{M_{ULT}}{FS} = \frac{21{,}311}{2}$ = 10,656 ft-lb

Lateral Loading in Cohesive Soil:

Refer to Section 5-6.03B, *Lateral Loading in Cohesive Soil*.

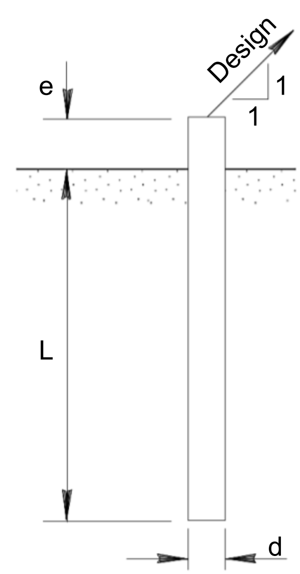


Figure D-29-4. Pile Lateral Loading in Cohesive Soil

Given Information

Unit weight of concrete:
$\gamma_c = 145$ pcf

Unit weight of soil:
γ_s = 110 pcf

Pile:
L = Depth below ground = 8 ft
e = Length from ground surface to ultimate lateral load = 2 ft
d = Pile diameter = 18 in = 1.5 ft

Undrained shear strength:
C_u = 1000 psf

Single use loading (FS = 2)

Determine allowable loading for poured-in-place concrete pile in Cohesive Soil

Working load value for lateral load H

L/d = 8/1.5 = 5.33   e/d = 2/1.5 = 1.33

Use Figure 5-24 to find $\frac{H_{ULT}}{C_u d^2}$

$\frac{H_{ULT}}{C_u\, d^2} \approx 5.5$ when e = 2'–0″

$H_{ULT} = 5.5 \times C_u d^2 = 5.5 \times (1{,}000)\,(1.5)^2 = 12{,}375$ lbs

Working Load Value for H = $\frac{H_{ULT}}{FS} = \frac{12{,}375}{2}$ = 6188 lbs

Working load value for moment M

$$f_c = \frac{H_{ULT}}{9C_u d} \qquad (5\text{-}6.03B\text{-}1)$$

$$= \frac{12{,}375}{9\,(1{,}000)(1.5)} = 0.917 \text{ ft}$$

$$M_{ULT} = H_{ULT}(e + 1.5d + 0.5f_c) \qquad (5\text{-}6.03B\text{-}2)$$

$$= (12{,}375)\,[2 + 2.25 + 0.46] = 58{,}266 \text{ ft-lb}$$

Working Load Value for M = $\frac{M_{ULT}}{FS} = \frac{58{,}266}{2}$ = 29,133 ft-lb

Concrete Stress:

Refer to Section 5-6.04, *Concrete Stresses,*

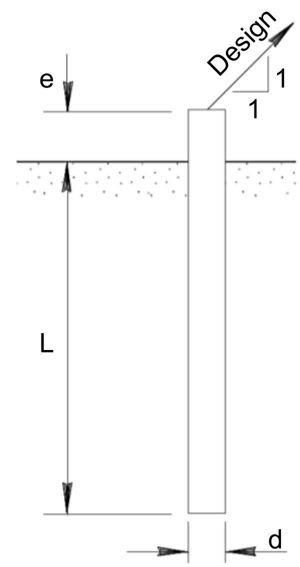


Figure D-29-5. Pile Lateral Loading for Concrete Stress

Given Information

Pile:
L = Depth below ground = 8 ft
e = Length from ground surface to ultimate lateral load = 2 ft
d = Pile diameter = 18 in = 1.5 ft

Unit weight of concrete:
$\gamma_c = 145 \text{ pcf}$

Concrete compressive strength:
f'_c = 3250 psi

Design loads:
V_{MAX} = 6188 lbs
H_{MAX} = 6188 lbs
M_{MAX} = 29,133 ft-lb

Single use loading (FS = 2)

Determine the concrete stress for this poured-in-place pile

With forces acting through the center of the pile consider one half of pile in compression.

Use the simplified equation:

$$f_c = \frac{Md}{2I_g} - \frac{V'}{A_g} \leq \frac{f'c}{2} \qquad (5\text{-}6.04\text{-}1)$$

where V' = 6188 lbs minus the pile weight above the plane of zero shear.

Distance to plane of zero shear $\approx \frac{M_{ULT}}{H_{ULT}} \approx \frac{M_{MAX}}{H_{MAX}} \approx \frac{29{,}133}{6188}$ = 4.7 ft

Pile Weight = $\pi\left(\frac{d}{2}\right)^2 (4.7 + 2)\,\gamma_c$

$= \pi\left(\frac{1.5}{2}\right)^2 (6.7)(145)$ = 1717 lbs

V' = 6188 – 1717 = 4471 lbs

$$I_g = \frac{\pi}{4}\left(\frac{d}{2}\right)^4 = \frac{\pi}{4}\left(\frac{18}{2}\right)^4 = 5153 \text{ in}^4$$

$$A_g = \pi\left(\frac{d}{2}\right)^2 = \pi\left(\frac{18}{2}\right)^2 = 254.5 \text{ in}^2$$

$$f_c = \frac{(29{,}133 \text{ x } 12)(18)}{2\,(5153.0)} - \frac{4{,}471}{254.5} = 593 \text{ psi} \leq 1625 \text{ psi} = \frac{f'c}{2}$$

Bar Reinforcing Stress:

Refer to Section 5-6.05, *Bar Reinforcing Stresses*.

Given Information

Pile:
- L = Depth below ground = 8 ft
- e = Length from ground surface to ultimate lateral load = 2 ft
- d = Pile diameter = 18 in = 1.5 ft

Bar Reinforcing Steel:
- 2-#8 Grade 60 bars, full length, 2" clear

Placed symmetrically along the pile axis

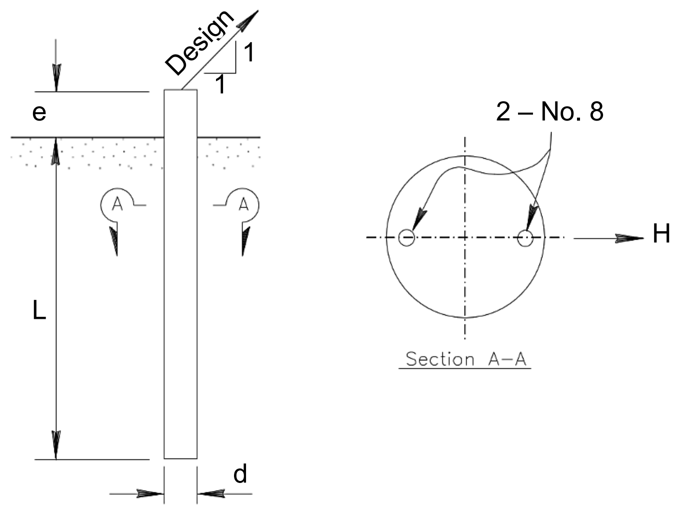


Figure D-29-6. Laterally Loaded Pile with Reinforcement

Design loads:
- V_{MAX} = 6188 lbs
- H_{MAX} = 6188 lbs
- M_{MAX} = 29,133 ft-lb

Single use loading (FS = 2)

Determine the bar reinforcing stress in this pile

$d_s = d_{pile}$ – 2[2″ clear] – 2(d_{bar}/2) = 18 – 2(2) – 2(1.0/2) = 13 in

A_s = 0.79 in^2
ΣA_s = 2(0.79 in^2) = 1.58 in^2

V' = V_{MAX} – pile weight = 6188 – 1717 = 4471 lbs

$$f_s = \frac{M}{A_s d_s} + \frac{V'}{\Sigma A_s} \qquad (5\text{-}6.05\text{-}3)$$

$$= \frac{29{,}133\ (12)}{(0.79)\ (13)} + \frac{4{,}471}{1.58} = 36{,}870 \text{ psi}$$

$$F_s \leq 0.70\ F_y = 0.7\ (60{,}000) = 42{,}000 \text{ psi} \qquad (5\text{-}6.05\text{-}4)$$

36,870 psi < 42,000 psi allowable **OK**

Combined Uplift and Horizontal Load:

Refer to Section 5-6.06, *Resistance to Combined Uplift and Horizontal Load*.

Given Information

Load capacities:
V_{ULT} = 15,800 lbs
H_{ULT} = 11,900 lbs

Single use loading (FS = 2)

Determine the load that the following pile types would be designed to resist:

a. For a plumb pile with load 30º from horizontal?
b. For a pile that is battered 15º towards the load with load 45º from H?

a. Plumb Pile

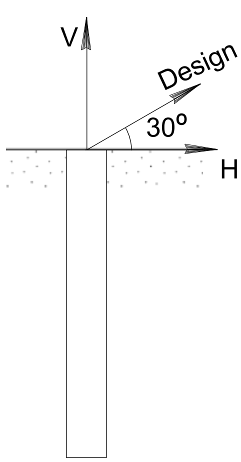


Figure D-29-7. Combined Loading for Plumb Pile

$$\text{Design}_1 = \frac{15{,}800}{\sin(30°)} = 31{,}600 \text{ lbs}$$

$$\text{Design}_2 = \frac{11{,}900}{\cos(30°)} = 13{,}741 \text{ lbs}$$

The design loading of 13,741 lbs governs

$$\text{Design working load} = \frac{13{,}741}{2} = 6871 \text{ lbs}$$

b. Battered Pile

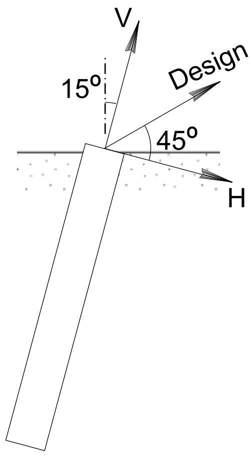


Figure D-29-8. Combined Loading for Battered Pile

$$\text{Design}_1 = \frac{15{,}800}{\sin(45°)} = 22{,}345 \text{ lbs}$$

$$\text{Design}_2 = \frac{11{,}900}{\cos(45°)} = 16{,}829 \text{ lbs}$$

The design loading of 16,829 lbs governs

$$\text{Design working load} = \frac{16{,}829}{2} = 8415 \text{ lbs}$$

The forgoing equations may be used when the horizontal force H is to be less than the computed ultimate lateral force H_{ULT}.

Appendix D Example 30 – Short Poured-In-Place Concrete Piles

Refer to *Falsework Manual,* Section 5-6, *Short Poured-In-Place Concrete Piles* and the sample calculations in Appendix D Example 29 – *Short Poured-In-Place Concrete Piles*. This example demonstrates how to perform a complete analysis for a short poured-in-place concrete pile.

Given Information

A contractor proposes to use an 18-inch diameter poured in place concrete pile as an anchorage for his falsework cable bracing. Prior to being used for bracing the falsework this pile will be used as an anchorage for the column reinforcing cage and form.

This anchor pile will be subjected to three short term loads in the same direction.

Soil (cohesionless):
Internal Friction $\emptyset$ = 35°
Unit weight $\gamma_s = 110$ pcf

Concrete:
Unit weight γ_c = 145 pcf
Compressive strength f'_c = 3250 psi

Bar Reinforcing Steel:
2-#5 Grade 60 bars, full length each side of centerline

Pile Dimensions:
a = 2 in.
b = 6 ¼ in
d = 18 in = 1.5 ft
e = 1.2 ft
θ = 35°
L = 12 ft (lower 2 ft submerged)

Design load:
P = 8000 lbs

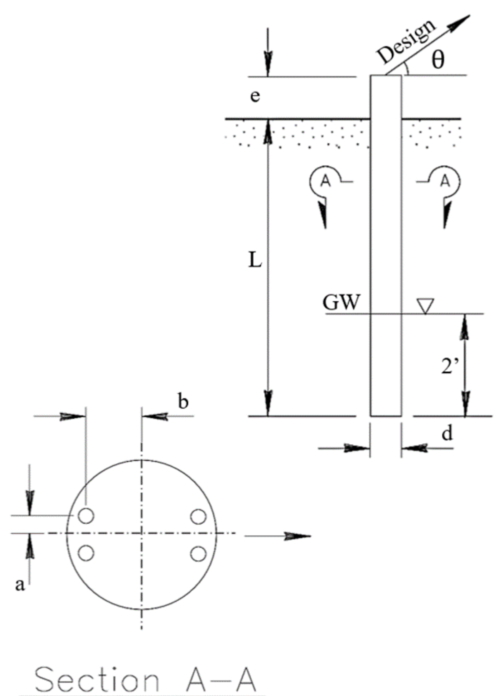


Figure D-30-1. Laterally Loaded Reinforced Concrete Pile

Determine the Adequacy of the Pile

Check for adequate reinforcement clearance

Find distance from center of pile to center of bar (use geometry) and bar radius

$$9 \text{ in} - \left\{\sqrt{(6.25)^2 + (2)^2} + \frac{0.625}{2}\right\} = 2.13 \text{ in} > 2 \text{ in minimum clearance} \quad \underline{\textbf{OK}}$$

Calculate load components

H_{DESIGN} = Design cos 35° = (8000) (cos (35°)) = 6553 lbs

V_{DESIGN} = Design sin 35° = (8000) (sin (35°)) = 4589 lbs

Calculate Factor of Safety

FS = 2.0 + (x-1) (0.25) = 2.0 + (3-1) (0.25) (5-6.03-1)

= 2.5 for lateral soil loading

Check Uplift Capacity

$$S = \beta\sigma_z$$

where:

z = 12 ft (z_{dry} = 10 ft; Z_{wet} = 2 ft)

σ_z = 10(110) + 2(110 – 62.4) = 1195 psf

$\beta = 1.5 - 0.315\, z^{1/2}$ = 1.5 – 0.315$(12)^{1/2}$ = 0.41 > 0.25 **OK**

S = 0.41 (1,195) = 490 psf ≤ 4,000 psf **OK**

Net pile shearing resistance $R_s = \pi d z S = \pi$(1.5)(12)(490) = 27,709 lbs

Pile weight $W = \pi \left(\frac{d}{2}\right)^2 L_p \gamma_c = \pi \left(\frac{1.5}{2}\right)^2 (12 + 1.2)(145)$ = 3382 lbs

Ultimate load capacity R = 27,709 + 3382 = 31,091 lbs

Working load $V = \frac{31{,}091}{2.5}$ = 12,436 lbs > 4,589 lbs **OK**

Check lateral capacity

4d = (4) (1.5) = 6.0 ≤ 12 ft (Meets minimum embedment length requirements)

$$\mathrm{K_p} = \tan^2\left(45° + \frac{\phi}{2}\right) = \tan^2\left(45° + \frac{35°}{2}\right) = 3.69$$

L/d = 12/1.5 = 8.0 ≤ 20 (meets short pile criteria) e/d = 1.2/1.5 = 0.8

From Figure 5-23, $\frac{\mathrm{H_{ULT}}}{\mathrm{K_p}\gamma_s \mathrm{d}^3} \approx 16$

Find effective unit weight of soil by using weighted average to account for variable soil layers:

$$\gamma_2 = \frac{(10)(110) + (2)(110-62.4)}{12} = 99.6 \text{ pcf}$$

$\mathrm{H_{ULT}}$ = 16 x $\mathrm{K_p}\gamma_s \mathrm{d}^3$ = 16 x (3.69)(99.6)(1.5)^3 = 19,846 lbs

Working Load Value for $\mathrm{H_{ULT}}$ = $\frac{19,846}{2.5}$ = 7938 lbs > 6533 lbs **OK**

$$(\mathrm{f_g})^2 = \frac{\mathrm{H_{ULT}}}{1.5\,\gamma_s\,\mathrm{dK_p}} \qquad (5\text{-}6.03\text{A-}1)$$

$$\mathrm{f_g} = \left(\frac{\mathrm{H_{ULT}}}{1.5\,\gamma_s\,\mathrm{dK_p}}\right)^{\frac{1}{2}} = \left(\frac{19,846}{1.5\,(99.6)(1.5)(3.69)}\right)^{\frac{1}{2}} = 4.90 \text{ ft}$$

$$\mathrm{M_{ULT}} = \mathrm{H_{ULT}}\left(\mathrm{e} + \frac{2\mathrm{f_g}}{3}\right)$$

$$= (19,846)\left(1.2 + \frac{(2)(4.90)}{3}\right) = 88,645 \text{ ft-lb}$$

Working Load Value for $\mathrm{M_{ULT}}$ = $\frac{88,645}{2.5}$ = 35,458 ft-lb

Pile Adequacy

Pile capacity is to be based on design loads. The lateral force $\mathrm{H_{DESIGN}}$ may be substituted for $\mathrm{H_{ULT}}$ and $\mathrm{M_{DESIGN}}$ for $\mathrm{M_{ULT}}$ in the critical soil equations.

V_{DESIGN} = 4,589 lbs

H_{DESIGN} = 6,553 lbs

$$(f_g)^2 = \frac{H_{ULT}}{1.5\ \gamma_s\ dK_p} \qquad (5\text{-}6.03A\text{-}1)$$

$$f_g = \left(\frac{H_{ULT}}{1.5\ \gamma_s\ dK_p}\right)^{\frac{1}{2}} = \left(\frac{6553}{1.5\ (99.6)(1.5)(3.69)}\right)^{\frac{1}{2}} = 2.82 \text{ ft}$$

$$M_{DESIGN} = M_{ULT} = H_{ULT}\left(e + \frac{2f_g}{3}\right) \qquad (5\text{-}6.03A\text{-}2)$$

$$M_{DESIGN} = 6553\left(1.2 + \frac{(2)(2.82)}{3}\right) = 20{,}183 \text{ ft-lb}$$

Depth to plane of zero shear of pile $\approx \frac{M_{DESIGN}}{H_{DESIGN}} \approx \frac{20{,}183}{6{,}553} = 3.08$ ft

Concrete Stress

Pile weight = $\pi\left(\frac{d}{2}\right)^2 (3.08 + e)\,\gamma_c = \pi\left(\frac{1.5}{2}\right)^2(3.08+1.2)(145) = 1096$ lbs

V' = 4589 – 1096 = 3493 lbs

$$I_g = \frac{\pi d^4}{64} = \frac{\pi(18)^4}{64} = 5153 \text{ in}^4$$

$$A_g = \frac{\pi d^2}{4} = \frac{\pi(18)^2}{4} = 254.5 \text{ in}^2$$

$$f_c = \frac{Md}{2I_g} - \frac{V'}{A_g} \leq \frac{f'c}{2} \qquad (5\text{-}6.04\text{-}1)$$

$$f_c = \frac{(20{,}183)(12)(1.5)(12)}{(2)(5153)} - \frac{3493}{254.5} = 409 \text{ psi} < 1625 \text{ psi} = \frac{f'_c}{2}$$ **OK**

$$V_u = 2\sqrt{f'_c} = 2\sqrt{3250} = 114 \text{ psi}$$

$$v_u = \frac{v}{0.5bd} \approx \frac{v}{0.5A} \approx \frac{6{,}553}{(0.5)(254.5)} \approx 51 \text{ psi}$$

51 psi < 114 psi allowable **OK**

Bar Reinforcing Stress

$A_s = 0.31\ in^2$ $d_{bar} = 0.625\ in$

$d_s = 2b = (2)(6.25) = 12.50\ in$

$\Sigma A_s = 4(0.31\ in^2) = 1.24\ in^2$

$$f_s = \frac{M}{A_s d_s} + \frac{V'}{\Sigma A_s} = \frac{(20{,}183)(12)}{(2 \text{ x } 0.31)(12.50)} + \frac{3493}{1.24} = 34{,}068 \text{ psi}$$

$F_s \leq 0.70\ F_y = 0.7\ (60{,}000) = 42{,}000$ psi (5-6.05-4)

34,068 psi < 42,000 psi allowable **OK**

This pile is capable of resisting the applied loads. The pile is satisfactory for use as designed by the contractor.

Appendix D Example 31 – Falsework Removal with Winches

This example shows the typical review items for falsework removal with winches. These items include: static stability – winch overturning & sliding, and deck stress analysis. The internal workings of the winch are not reviewed here. It is assumed that the weight of the falsework being removed has been previously calculated.

Winches are typically laid perpendicular to the bridge centerline (not always though, sometimes other configurations are required to accommodate various geometry) and can be supported with one, two, three, or more supports.

Given Information

This example uses a two-support winch as shown here:

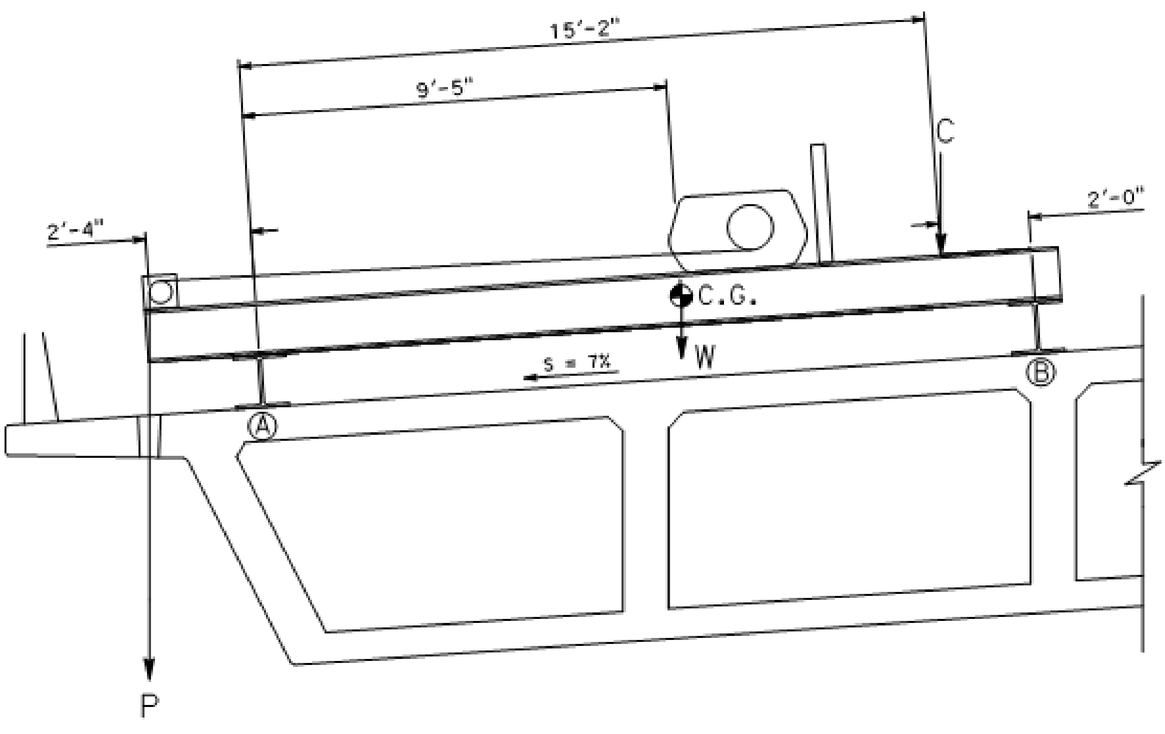


Figure D-31-1. Typical Winch with Two Supports.

Where $\mathrm{P} = \text{design load supported by the winch}$

$\mathrm{W} = \text{weight of winch through the center of gravity}$

$$\mathrm{C} = \text{weight of counterweights}$$

The dimensions and winch weights (C and W) must be provided by the contractor with the submittal. The design load, P, is the weight of the falsework being supported and must be calculated independently by the reviewer for the worst case scenario. For this example the following values are given for the analysis:

$\mathrm{P} = 32 \text{ kips}$

$\mathrm{W} = 5{,}000 \text{ lbs} = 5.0 \text{ kips}$

For C, use three $4' \text{ x } 4' \text{ x } 1'$ reinforced concrete deadmen ($\gamma_{\text{concrete}} = 150 \text{ pcf}$)

$$\mathrm{C} = 3\,(4 \text{ ft} * 4 \text{ ft} * 1 \text{ ft})(150 \text{ pcf}) = 3\,(16 \text{ ft}^3)(150 \text{ pcf}) = 7{,}200 \text{ lbs}$$

$$\mathrm{C} = 7.2 \text{ kips}$$

Check Overturning

The *2018 Standard Specifications,* Section 48-2.02B(1), *Falsework – Materials – Design Criteria – General,* requires that the load used for analysis of the overturning moment and sliding of the winch system must be 150% of the design load (Note: Cross slope is ignored for moment calculations since there is minimal influence on the moments in this example).

$$\text{Analysis load} = 1.5 * \mathrm{P} = 1.5 * 32 \text{ kips} = 48 \text{ kips}$$

Estimate overturning moment $\mathrm{M_{OT}}$:

$$\mathrm{M_{OT}} = (48 \text{ kips})(2.33 \text{ ft}) \approx 112^{\text{ft}-\text{kip}}$$

Note: The moment is taken at support A in Figure 1 since this is the point of rotation for the winch system.

Estimate the resisting moment $\mathrm{M_r}$:

$$\mathrm{M_r} = (5.0 \text{ kips})(9.42 \text{ ft}) + (7.2 \text{ kips})(15.17 \text{ ft}) \approx 156^{\text{ft}-\text{kip}}$$

Check demand vs. capacity:

$$\mathrm{M_{OT}} \leq \mathrm{M_r} \Rightarrow 112 \text{ kip} \cdot \text{ft} < 156^{\text{ft}-\text{kip}} \qquad \textbf{OK}$$

$$\frac{\text{Demand}}{\text{Capacity}} = \frac{\mathrm{M_{OT}}}{\mathrm{M_r}} = \frac{112^{\text{ft}-\text{kip}}}{156^{\text{ft}-\text{kip}}} \approx 0.72 \leq 1.00 \qquad \textbf{OK}$$

Check Sliding

As with overturning, sliding requires the load for analysis to be increased by 150% per the *2018 Standard Specifications.*

First, determine the driving force for the sliding, f_s:

$$f_s = (1.5P + W + C)\sin\theta, \text{where } \theta = \tan^{-1}\left(\frac{s}{100}\right) = \tan^{-1}\left(\frac{7}{100}\right) \approx 4^{\circ}$$

$$f_s = (1.5 * 32 \text{ kips} + 5.0 \text{ kips} + 7.2 \text{ kips})\sin(4) \approx 4 \text{ kips}$$

The next step in checking the winch sliding is to determine the support reactions at supports A & B to find sliding resistance. Since the supports are where the resistance to sliding occurs, do not apply the 150% analysis increase. The reactions are:

$$R_A = \frac{\overbrace{(32 \text{ kips})}^{P}(19.5 \text{ ft}) + \overbrace{(5.0 \text{ kips})}^{W}(7.75 \text{ ft}) + \overbrace{(7.2 \text{ kips})}^{C}(2 \text{ ft})}{17.167 \text{ ft}}$$

$$\approx 39 \text{ kips}$$

$$R_B = 32 \text{ kips} + 5.0 \text{ kips} + 7.2 \text{ kips} - 39 \text{ kips} \approx 5.2 \text{ kips}$$

The sliding resistance F_r is:

$$F_r = \mu \sum F_N = \mu(R_A + R_B)\cos\theta$$

Where: $\mu = \text{coefficient of friction from steel to concrete} = 0.45$

(Note: the coefficient of friction can be as low as 0.10 depending on the condition of the surfaces and the presence of water or lubricants so use caution in determining the coefficient of friction. 0.45 was used here as a typical coefficient of friction between dry, rough concrete and weathered steel.)

$$F_r = \mu(R_A + R_B)\cos\theta = 0.45(39 \text{ kips} + 5.2 \text{ kips})\cos(4) \approx 19.8 \text{ kips}$$

Check demand vs. capacity:

$$\mathbf{f_s \le F_r \Rightarrow 4\ kips < 18\ kips} \qquad \mathbf{OK}$$

$$\frac{\text{Demand}}{\text{Capacity}} = \frac{f_s}{F_r} = \frac{4 \text{ kips}}{19.8 \text{ kips}} \approx 0.20 \le 1.00 \qquad \mathbf{OK}$$

Deck Stress Analysis

The bridge deck stress analysis is the next step in the review. The deck stress is reviewed for maximum moment and shear in the slab. Ideally, the winch supports would be placed directly on the girder lines, however this is rarely possible due to bridge geometry, falsework configuration, winch geometry, and skew. It is therefore necessary to examine the moment and shear at various locations to determine the maximum stress induced. Figure 2 shows a typical winch layout pattern with the winch supports on and adjacent to the girder lines as well as in between the girders.

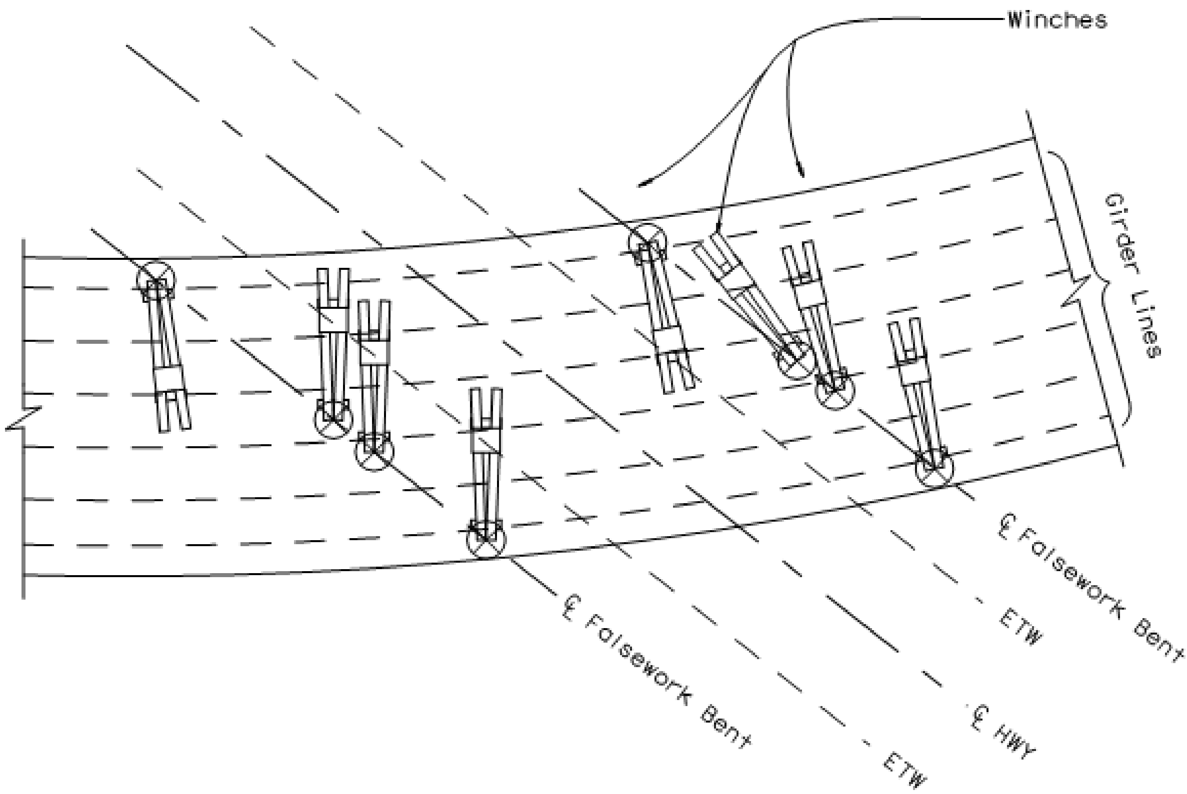


Figure D-31-2. Typical Winch Layout Plan.

Given the part bridge typical section shown in Figure 3, find design strength of the bridge deck for flexure and shear then compare to loads applied from winch system. The design code used for this analysis is the Bridge Design Specifications, LFD Version, April 2000 (BDS, LFD 2000). (A similar analysis can be performed using AASHTO LRFD Design Specifications or concrete design textbook).

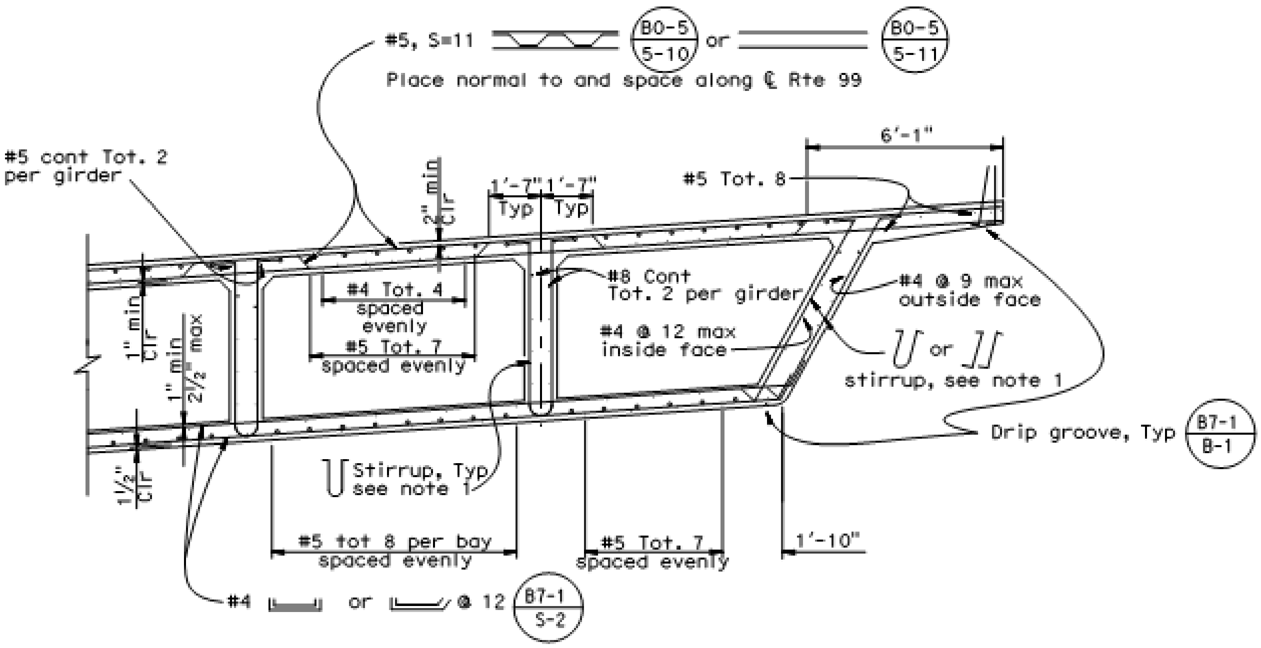


Figure D-31-3. Part typical section used in this example.

Flexural Strength

Given the following values:

$f'_c = 4{,}000 \text{ psi}$

$f_y = 60{,}000 \text{ psi}$

$A_s = 0.31 \text{ in}^2$ for #5 transverse reinforcement

Transverse reinforcement spacing, $s = 11$ in (Standard Plan B0-5 Detail 5-10 used in this example)

Deck thickness, $t = 9$ in

Girder centerline spacing $= 9$ ft

Find the factored flexural strength φM_n:

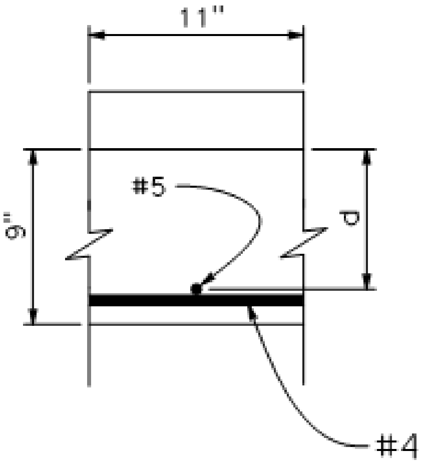


Figure D-31-4. Deck Transverse Strip Section (truss and top bar not shown for clarity).

As shown in Figure 4, a deck strip of width of 11 inches was used for this example. The 11 inches were taken from the typical section "s" value for the transverse rebar as shown. Only the bottom #5 tension reinforcement was used in these calculations. The capacity added by the truss bars is ignored but could be included in a more refined analysis.

Find dimension d from extreme compression fiber (top of deck) to tension bar c.g.:

$$\mathrm{d = 9\ in - 1\ in - 0.5\ in} - \frac{\mathrm{0.625\ in}}{2} \approx \mathrm{7.19\ in}$$

The design moment strength for rectangular sections with tension reinforcement only per BDS, LFD 2000 Section 8.16.3.2 Equation 8-16 is:

$$\varphi \mathrm{M_n} = \varphi \left[\mathrm{A_s f_y}\left(\mathrm{d} - \frac{\mathrm{a}}{2}\right)\right]$$

where:

$$\varphi = 0.90 \qquad \text{(BDS, LFD 2000 8.16.1.2.2 strenth reduction for flexure)}$$

$$\mathrm{a} = \frac{\mathrm{A_s f_y}}{\mathrm{0.85 f'_c b}} = \frac{(0.31\ \mathrm{in^2})(60{,}000\ \mathrm{psi})}{0.85(4{,}000\ \mathrm{psi})(11\ \mathrm{in})} = 0.4973\ \mathrm{in} \approx 0.5\ \mathrm{in}$$

$$\mathrm{b = width\ of\ strip = 11\ in}$$

Calculate the factored flexural strength $\varphi \mathrm{M_n}$:

$$\varphi M_n = 0.90\left[(0.31\ \text{in}^2)(60{,}000\ \text{psi})\left(7.19\ \text{in} - \frac{0.5\ \text{in}}{2}\right)\right] \approx 116{,}176^{\text{in-lb}} \approx 9.7^{\text{ft-kip}}$$

Maximum moment occurs when the winch support is placed at the center of a girder bay (mid-span of transverse deck slab). In this case, treat the maximum winch support load as a point load and assume the span is simple (divide moment by 5 for the support width and multiply by 11/12 to get the equivalent load on an 11 inch strip to compare to capacity). A diagram of the 11-inch strip, support load, and the distribution of the moment is shown in Figure 5.

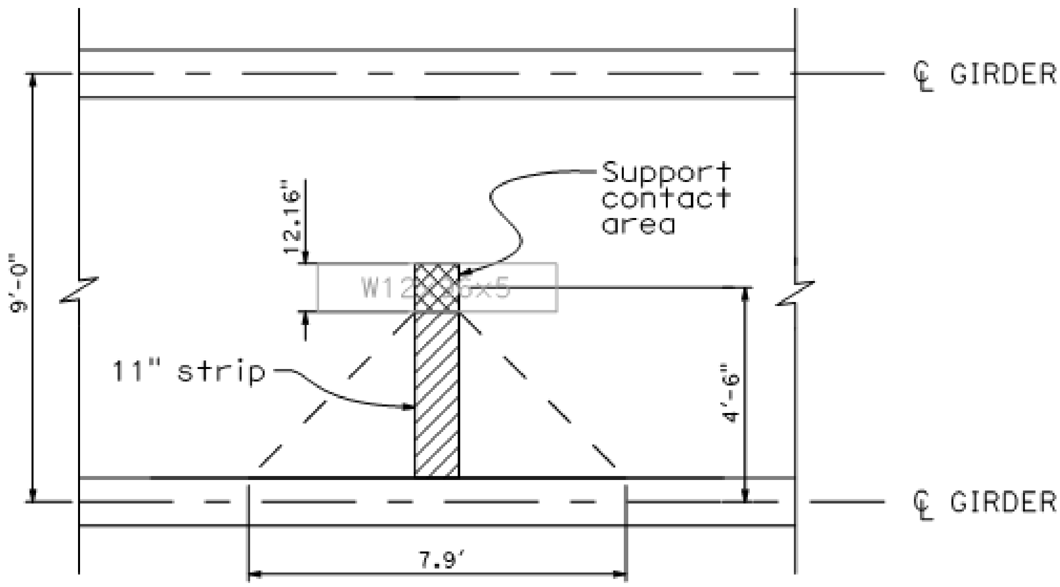


Figure D-31-5. Strip used for Flexure Analysis and Distribution of Moment.

$$M_{max} = \frac{R_A l}{4} = \frac{(39\ \text{kips})(9\ \text{ft})}{4} \approx 87^{\text{ft-kip}} \quad \Rightarrow \quad \frac{87^{\text{ft-kip}}}{5\ \text{ft}}\left(\frac{11}{12}\right) \approx 16.0^{\text{ft-kip}}$$

For a one-way slab, this moment can be distributed approximately 45° to the supports. Which is:

$$= 11\ \text{in} * \frac{1\ \text{ft}}{12\ \text{in}} + 2\left(4.5\ \text{ft} - \frac{1\ \text{ft}}{2} - \frac{12.16\ \text{in}}{2}\frac{1\ \text{ft}}{12\ \text{in}}\right) \approx 7.9\ \text{ft}$$

$$M_{max} = \frac{16.0^{\text{ft-kip}}}{7.9\ \text{ft}} \approx 2^{\text{ft-kip}}$$

Check demand vs. capacity:

$$\mathbf{M_{max} \le \varphi M_n \Rightarrow 2 < 9.7} \qquad \textbf{OK}$$

$$\frac{\text{Demand}}{\text{Capacity}} = \frac{M_{max}}{\varphi M_n} = \frac{2\ \text{kip}\cdot\text{ft}}{9.7\ \text{kip}\cdot\text{ft}} \approx 0.21 \leq 1.00 \qquad \textbf{OK}$$

Shear Strength

Reference: BDS, LFD 2000 Section 8.16.6 – Shear.

Per Section 8.16.6.1, the nominal shear strength V_n is:

$$V_n = V_c + V_s \qquad \text{Equation 8-47}$$

where:

$$V_c = 2\sqrt{f_c'}b_w d \qquad \text{Equation 8-49 (Nominal shear strength of concrete)}$$

$$V_s = 0 \text{ since there is no shear reinforcement in slab}$$

First, the two-way punching shear of the winch support acting at the mid span of the transverse deck slab is checked. The punching shear perimeter for the W12 x 96 x 5 winch support is illustrated in Figure 6.

Calculate the effective perimeter, b_o, and use this value for b_w:

$$b_o = 2\overbrace{\left\{2\left(\frac{d}{2}\right) + b_f\right\}}^{\text{effective width}} + 2\overbrace{\left\{2\left(\frac{d}{2}\right) + l\right\}}^{\text{effective length}}$$

where

$$b_f = \text{width of support} = 12.16\ \text{in}$$

$$l = \text{length of support} = 5\ \text{ft} = 60\ \text{in}$$

$$d = 7.19\ \text{in (see page 6 of 10 for calculations)}$$

$$b_o = 2\left\{2\left(\frac{7.19\ \text{in}}{2}\right) + 12.16\ \text{in}\right\} + 2\left\{2\left(\frac{7.19\ \text{in}}{2}\right) + 60\ \text{in}\right\} \approx 173.1\ \text{in}$$

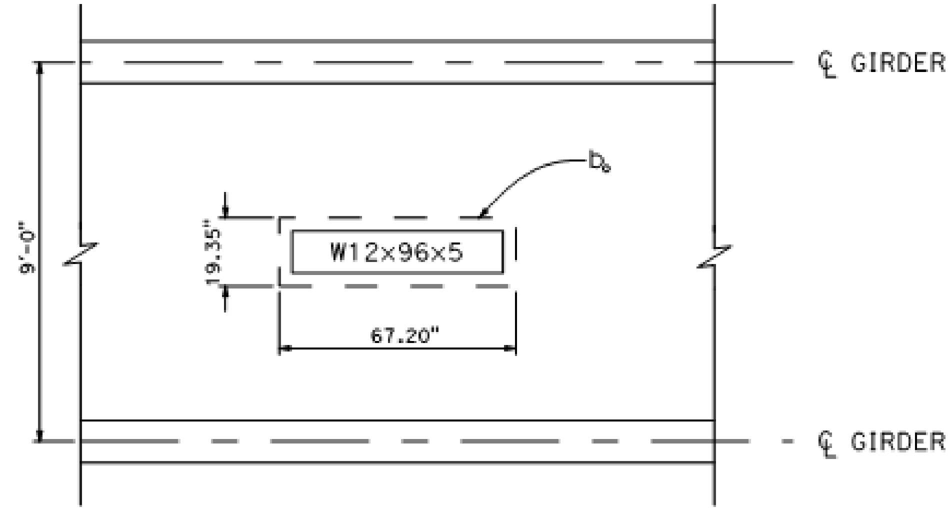


Figure D-31-6 – Two-Way Punching Shear at Mid-Span of Transverse Deck Slab.

Calculate the factored shear φV_n where $\varphi = 0.85$ per BDS LFD 2000 Section 8.16.1.2.2:

$$\varphi V_n = \varphi(V_c) = \varphi\left(2\sqrt{f_c'}b_o d_{eff}\right) = 0.85\left(2\sqrt{4000}\text{ psi} * 173.1\text{ in} * 7.19\text{ in}\right) \approx 133{,}815\text{ lbs} = 133.8\text{ kips}$$

Compare the two-way punching shear capacity to the maximum support load:

$$\mathbf{R_A \leq \varphi V_n \Rightarrow 39\ kips} < 133.8\text{ kips} \qquad \textbf{OK}$$

$$\frac{\text{Demand}}{\text{Capacity}} = \frac{R_A}{\varphi V_n} = \frac{39\text{ kips}}{133.8\text{ kips}} \approx 0.29 \leq 1.00 \qquad \text{OK}$$

Next, calculate the one-way shear with the support at mid-span and 1 foot from girder centerline shown in Figure D-31-7.

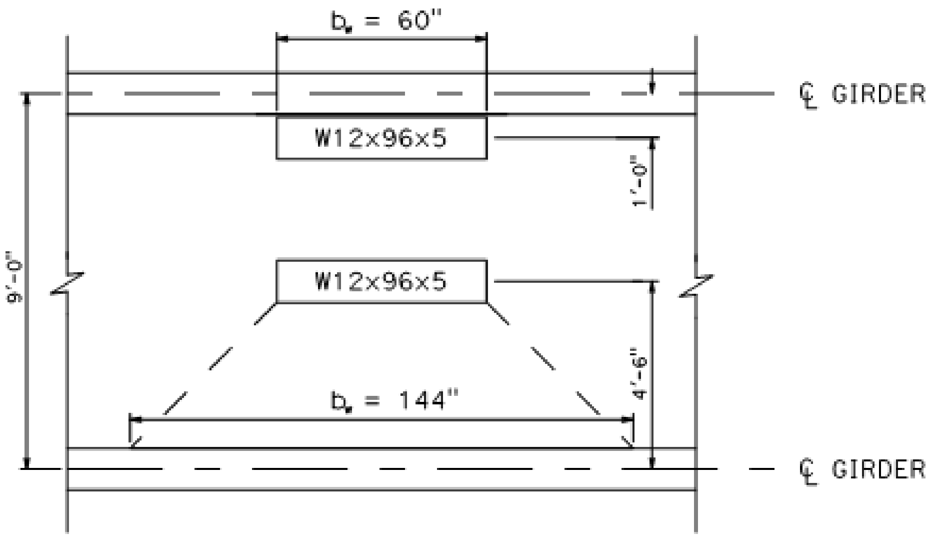


Figure D-31-7. One-Way Shear at Mid-Span and Near Girder of Transverse Deck Slab.

Mid-span one-way shear capacity:

$$\varphi V_n = \varphi\left(2\sqrt{f_c'}b_w d\right) = 0.85\left(2\sqrt{4000}\text{ psi} * 144\text{ in} * 7.19\text{ in}\right) \approx 111{,}319\text{ lbs} \approx 111.3\text{ kips}$$

$$\mathbf{R_A \leq \varphi V_n \Rightarrow 39\text{ kips} < 111.3\text{ kips} \qquad OK}$$

$$\frac{\text{Demand}}{\text{Capacity}} = \frac{R_A}{\varphi V_n} = \frac{39\text{ kips}}{111.3\text{ kips}} \approx 0.35 \leq 1.00 \qquad \mathbf{OK}$$

Capacity adjacent to girder:

$$\varphi V_n = \varphi\left(2\sqrt{f_c'}b_w d\right) = 0.85\left(2\sqrt{4000}\text{ psi} * 60\text{ in} * 7.19\text{ in}\right) \approx 46{,}383\text{ lb} \approx 46\text{ kips}$$

$$\mathbf{R_A \leq \varphi V_n \Rightarrow 39\text{ kips} < 46\text{ kips} \qquad OK}$$

$$\frac{\text{Demand}}{\text{Capacity}} = \frac{R_A}{\varphi V_n} = \frac{39\text{ kips}}{46\text{ kips}} \approx 0.85 \leq 1.00 \qquad \mathbf{OK}$$