FORMS AND FALSEWORK
5-393.200
(Note: This section uses English units only)

5-393.201 Introduction

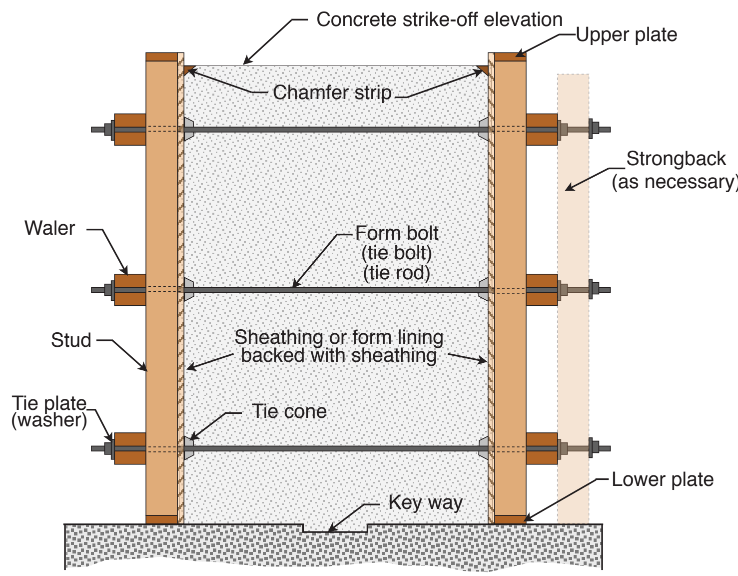
This Chapter is intended to be a resource for project engineers to help them develop a basic understanding of the design, construction, and performance of forms and falsework. Structural concrete is a major constituent used on almost every highway bridge. That use has become more complex due to two trends in bridge construction. First, the practice of bridge design has benefited from advances in structural engineering allowing for the creation of intricate and complicated structures. Secondly, great advances have been in the material properties of what has been termed high performance concrete.

The incorporation of these new advances in heavy concrete construction for bridges requires that the forms and falsework must be designed to meet the new demands from the designs. Forms and falsework for concrete structures have a significant impact on the cost, time and quality of the completed structure. Each structure is unique and requires forms and falsework that faithfully implement the intent of the bridge designer.

A form is generally described as those members, usually set vertically, to resist the fluid pressure from the plastic concrete to maintain its desired shape until it has set up. See **Figure 5-393-200-1** for a cross sectional view of a typical form. Falsework is a temporary structure used to support work in the process of construction, in concrete construction, it is the framework required to maintain a concrete unit in the desired position until it achieves sufficient strength to carry its own weight. See **Figure 5-393-200-2** for an elevation view of timber bent used as falsework for support for a concrete slab span bridge under construction.

The word formwork will be used in the broadest sense to include the total system of support for freshly placed concrete. Falsework supports concrete not resting on earth or previously cast concrete. There are some situations in concrete bridge construction where some elements in the forms and falsework serve both functions simultaneously. See **Figure 5-393-200-3** for details of the forms and falsework for a concrete pier cap where the division between members that are considered forms and those that are considered falsework is not immediately evident. The distinction between elements that are considered form or falsework does not impact the need for structural analysis of all members. The level of skill required to produce a good formwork system is as important as the level of skill required to produce the right combination of steel and concrete for the structural system for the bridge.

Many of the esthetic considerations of modern highway bridges are created directly by features incorporated into the forms. The geometry and fidelity of the lines and surfaces of cast-in-place concrete of the completed structure are dependent on the quality of the design and workmanship of the forms and falsework.

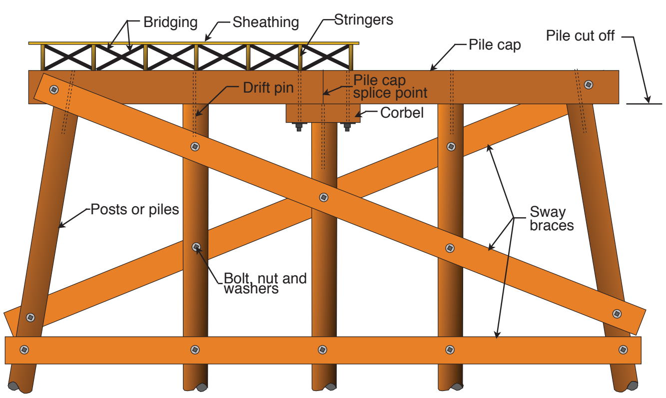


CROSS SECTION OF TYPICAL VERTICAL FORMS

Figure 5-393-200-1– Details of a typical application of a vertical form. Plywood sheathing is attached to wall studs. The lateral pressure from the fresh concrete is resisted by the horizontal tie-bars or tie-rods. The tie-rods transfer the internal force to the horizontal walers.

For most structures, more time and cost are required to make, erect, and remove forms and falsework than the time and cost to place the concrete or reinforcing steel. For some structures, the cost of the forms and falsework exceeds the cost of the concrete and steel combined. The *Standard Specifications for Construction* contains the provisions for the design and performance of forms and falsework. The Contractor is responsible for the design, construction and performance of the forms and falsework. This arrangement allows the Contractor discretion in the design and construction of the forms and falsework. Performance-based specifications for formwork do not prohibit the Contractor from exercising ingenuity in the design, construction and economical selection of materials.

The structural design of the formwork for modern highway bridges can be a very complicated and intricate process. This chapter discusses a wide range of possible configurations of formwork. Some formwork applications do not warrant the use of all the possible refinements. Several simplifications of the design process can be used for modest sized structural applications. Most of those simplifications can be handled by making a couple of key assumptions to redefine the design conditions that actually control the final configuration. The key assumptions used must generate a more conservative result than the more comprehensive analysis calculations. It is the responsibility of the engineer designing the forms and falsework to determine if the simplifications contained in this manual are appropriate for the analysis of his design.



TYPICAL FALSEWORK PILE BENT SUPPORTED WITH DRIVEN PILES

Figure 5-393-200-2– Details of a typical timber pile bent used for concrete slab-span construction. The spacing between bents and the pile spacing within the bents are design variables dependent on project conditions.

This chapter contains an overview of the design process for forms and falsework. It is not intended to be an exhaustive discourse, rather it is intended to provide information about formwork in two general areas. First, there are examples of the typical application of forms and falsework that can be used on typical highway bridges. The types of form and falsework featured represent those that engineers may encounter on typical projects. Next, there is some basic technical information presented. This information is used to support the design examples given at the end of the chapter. This technical information is only the minimum needed to work through and follow the example problems.

At the end of the chapter is a list of reference books and other sources for those users who want additional detailed information. There is also a glossary containing words unique to forms and falsework and a list of the symbols and notations used along with the appropriate units of measure at the end of this chapter.

5-393.202 General Requirements

Forms are required for all cast-in-place concrete except portions of footings that extend into solid rock. Concrete is never to be cast against the side of an earth excavation. Concrete not supported on earth or previously cast concrete must be supported by falsework. The need for forms and falsework is self-evident in every construction circumstance. This issue is not negotiable. All forms and falsework must be designed to resist all of the imposed loads and pressures without undo distortion.

A. Contractor Responsibilities

The Contractor is responsible for the design and construction of the forms and falsework necessary for the successful completion of the project as contained in the plans and specifications. All of the forms and falsework for bridges must be designed by a registered engineer. If requested by the Engineer, the Contractor must submit detailed design calculations and plan drawings certified by a Registered Engineer. The Engineer designing the forms and falsework will be considered the Responsible Engineer. Those calculations must comply with the requirements of the applicable provisions of the *Standard Specifications for Construction* as published by the Minnesota Department of Transportation. If requested by the Engineer, the Contractor shall submit a certification that the forms and falsework were constructed pursuant to the calculations and plans, and that the forms and falsework was inspected by the Responsible Engineer prior to the placement of any concrete.

It is the Contractor's responsibility to provide the Engineer the specifications and technical details including safe-load values for any new or unused system or device that they propose to use for formwork. This includes materials with unknown strength properties. It is the Contractor's responsibility to verify to the Engineer's satisfaction that the strength and safety of any device or system and the workability of the device or system can safely produce the desired end-product. This verification can be provided in the form of:

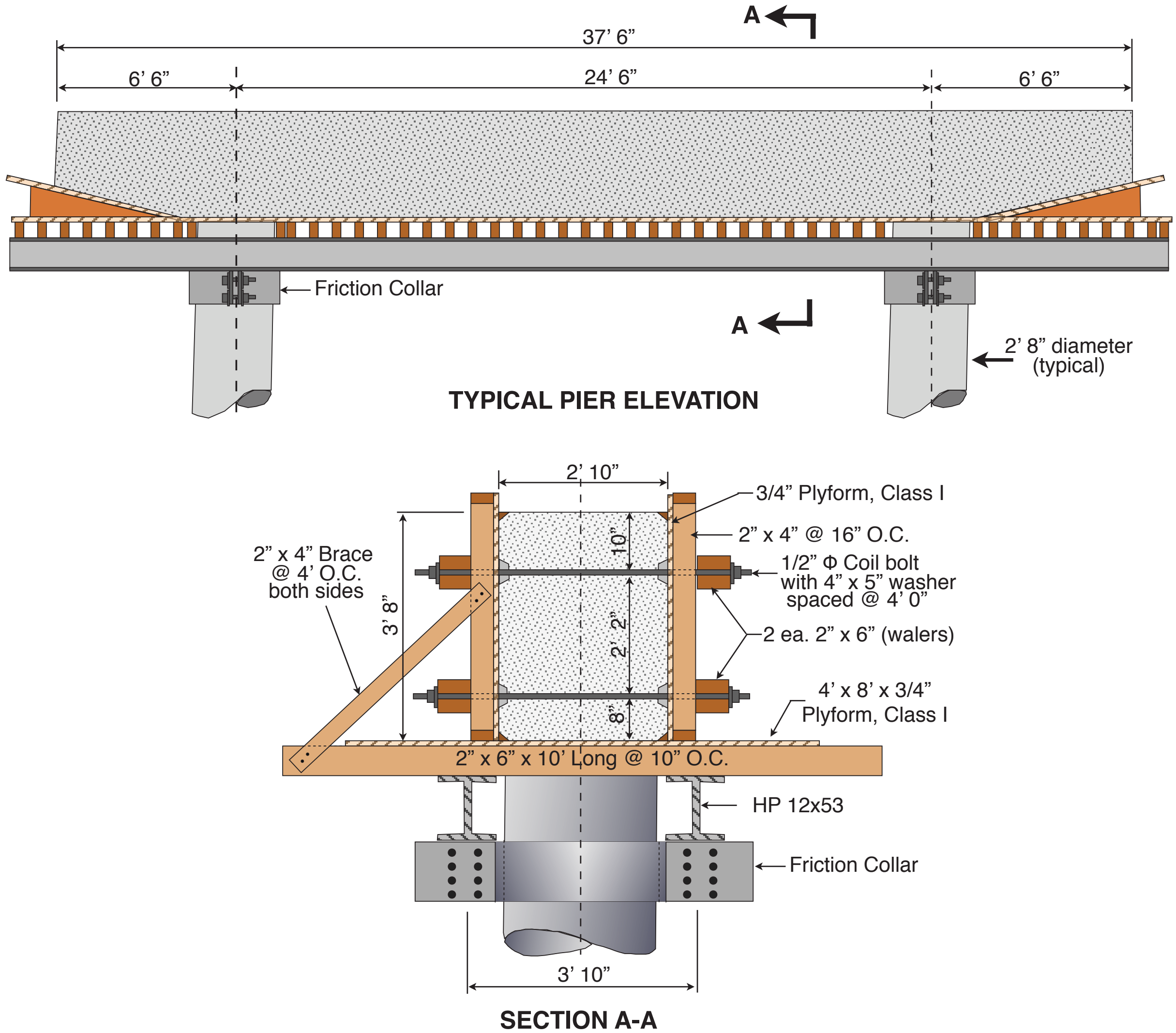


Figure 5-393-200-3– Details of typical form and falsework used for the construction of concrete pier caps.

1. Full scale field test
2. Tests by a reputable testing laboratory
3. Certified design calculations
4. Manufacturers literature
5. A combination of the above items

B. Engineer's Responsibilities

The Engineer's responsibilities with respect to the forms and falsework can be initiated in two general ways. First, the terms of the contact for certain levels of construction will explicitly require the Contractor to submit plans and design calculations for the formwork. The contract for those types of project may also explicitly require the Responsible Formwork Engineer to inspect the completed formwork prior to placement of concrete and certify that the forms and falsework have been constructed in accordance with the plans and calculation initially submitted by the Contractor.

In the other method, the Engineer may request calculations and plan details from the Contractor for the forms and falsework. Once the plans and calculations have been received by the Engineer, they should be reviewed as to strength, method of construction, safety, potential problems, and ability to produce the desired product. Approval to use such plans should be noted as being approved as to type of construction and should also bear a note that such acceptance is conditionally based on making changes that the Engineer has noted on the plans. When evaluating a new or untried device or system, approval (if given) should be given only on a performance basis. Such approval of plans does not relieve the

Contractor of the responsibility for results obtained by use of the plans (see *Standard Specifications for Construction* as published by the Minnesota Department of Transportation).

For certain types of structures, a review by the Contractor's Responsible Formwork Engineer is required prior to acceptance of the completed formwork. The Engineer should be present during this review of the formwork. No use of the formwork should be permitted until the Responsible Formwork Engineer has completed the review and has authorized its use. This authorization should be in the form of a written certification that formwork has been constructed in compliance with the original design calculations and plans.

A continuing inspection should be made during placement of forms and falsework members to assure conformance with the approved plans (if used), to assure structural soundness and accuracy, and to minimize the need for last minute corrections.

Concrete pours are to be made in accordance with approved pour sequences. Where approval of pour sequences is not required, pours should be as per the form or falsework design and should provide balanced loading to the extent possible. A follow-up inspection during and after concrete placement should be made to assure that the forms and/or falsework function as intended with regard to deflection, tolerances, etc.

5-393.203 Types of Forms and Falsework

The number of types, styles and configurations of concrete form and falsework is almost as numerous as there are concrete structures. Even with all of the possible variations, most applications can be conveniently placed into one of a few categories. The first natural division is form or falsework. This division is based on whether the work primarily confines the fresh concrete with a boundary, called a form, or whether the temporary work supports the fresh concrete from resting on the ground or on previously placed concrete, and is referred to as falsework. Additionally, most forms and falsework are referred to by names that are based on the actual structural element being created, such as pier cap or wing wall.

A. Formwork:

Most forms are vertical, and create a visible concrete surface in the finished structure. These forms are designed to resist the internal pressure produced by the fresh concrete plus any additional forces caused by or associated with the placement of the concrete. One of the most significant factors controlling the pressure exerted by the fresh concrete is the rate at which the form is filled with the concrete. This is usually referred to as the "rate-of-pour." The following are brief descriptions of the more common forms used in bridge construction.

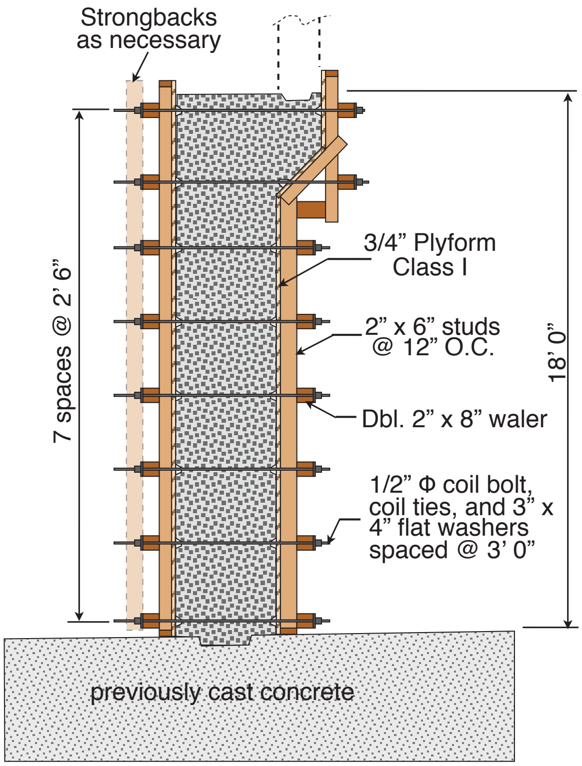
1. Vertical Forms: See **Figure 5-393-200-1**, page 5-393.200(1), for an illustration of a typical application of this type of construction. This type of form formwork has a sheathing normally employing a plywood-like product called Plyform, that actually comes in contact with the fresh concrete. That sheathing is attached to vertical wood members, called studs and thus form the walls of the completed form. The horizontal pressure from the fresh concrete on these walls is resisted by horizontal tie-bars that terminate at a waler on each side of the element being cast. These rods or bars have nuts, washers, and bearing plates at each end that press against horizontal wood members, called walers. The walers normally consist of two members with a space between them to accommodate the tie bars, that bear against the studs. The structural stability of most forms is provided by the more or less equal pressure on each wall of the form. This requires that the forms be filled symmetrically with approximately equal depth of concrete on each wall as the form is filled with concrete.

2. Pier Cap Forms: A typical application of formwork is for concrete pier caps. See **Figure 5-393-200-3,** page 5-393.200(3), for an illustration of this type of formwork. This type of formwork also requires falsework. The sheathing under the cap serves as both form and falsework. It is analyzed as an element of the falsework.

3. Abutment Wall Forms: Concrete abutments require rather complicated formwork. The dimension and function of the various sections of the abutment change over the height of the abutment. See **Figure 5-393-200-4** for details of a typical abutment formwork. Most concrete is poured in stages with slightly different form designs for each stage that reflect the needs and requirements unique to each stage.

B. Falsework:

This type of temporary work is primarily designed to support weight. There are almost an infinite number of combinations of spans, weights, and materials. This multitude possibilities can be covered by only a few typical applications that represent the type of work that is used on a majority of highway bridges. Additionally, once the principals of design and construction of these applications are understood, they can be applied to a wide range of materials, components, and loads.



TYPICAL ABUTMENT MAIN WALL FORMS

Figure 5-393-200-4– Details of typical formwork used in concrete abutment construction.

1. Falsework Pile Bents: These are used to support the falsework for the construction slab-span concrete bridges. See **Figure 5-393-200-2** ,page 5-393.200(2), for details of a typical timber pile bent falsework.

2. Pier Cap Falsework: Concrete pier caps require both forms and falsework in their construction. A typical example of this type of construction can be seen in **Figure 5-393-200-3,** page 5-393.200(3).

3. Steel Beam — Typical Slab and Overhang Falsework: One of the most commonly used falsework is to support concrete deck construction. Concrete deck construction requires support for the fresh concrete between the girders and for the overhang outside of the fascia girder. See **Figure 5-393-200-5** for a few examples for this type of construction with steel beams.

4. **Concrete Beam — Typical Slab and Overhang Falsework:** Construction of bridges with prestressed concrete girders use many of the same construction techniques used for steel beam bridges, but with slightly different details. See **Figure 5-393-200-6** for typical construction details involving bridges with prestressed girders

5. **Slab-span Falsework:** There are several commonly accepted methods of supporting slab-span bridge construction. Several companies produce and sell tubular steel scaffolding and shoring systems that are used. One other method is to use driven piles, either steel or timber. See **Figure 5-393-200-2** for details of that type of construction.

6. **Additional Falsework Applications:** Bridge construction projects sometimes involve types of construction not covered by the above examples. These projects could include Box Girders, Tunnels and Tunnel Portals, Shoring associated with railroad crossings, and Cofferdam construction. These projects use specialized material and construction techniques. Engineers should contact the Minnesota Department of Transportation (MN/DOT) Bridge Office for information and assistance for these projects.

5-393.204 Forms and Falsework Materials

A. General:

Forms and falsework material described below are listed with either an allowable maximum working stress or a basis for determining the safe load. The working stress or allowable stress shown is based on the use of sound material for temporary construction. The word temporary, as used here, denotes the time-frame of use and should not be interpreted to infer any reflection on the quality of construction. In general, previously used material is permitted, provide it is in good condition.

B. Falsework Piling:

The material requirements for falsework piling are stated in the *Standard Specifications for Construction* as published by the Minnesota Department of Transportation. Maximum allowable loads, in tons, are shown in **Table 5-393-200-1**.

C. Lumber: General requirements for lumber for falsework and forms are specified in the appropriate provisions contained in *Standard Specifications for Construction.* Lumber that has been planed on a planing machine is said to be "dressed lumber or surfaced lumber." That planing or surfacing can be on either one side (S1S) or on two sides (S2S) or two edges (S2E) or any combination therefore including complete planing (S4S). See **Table 5-393-200-2** for a list of the nominal and surfaced dimensions of most common sizes of lumber.

Lumber that has not been surfaced or dressed is said to be "rough lumber." Lumber that was originally sawn to dimensions larger that the nominal dimensions so that

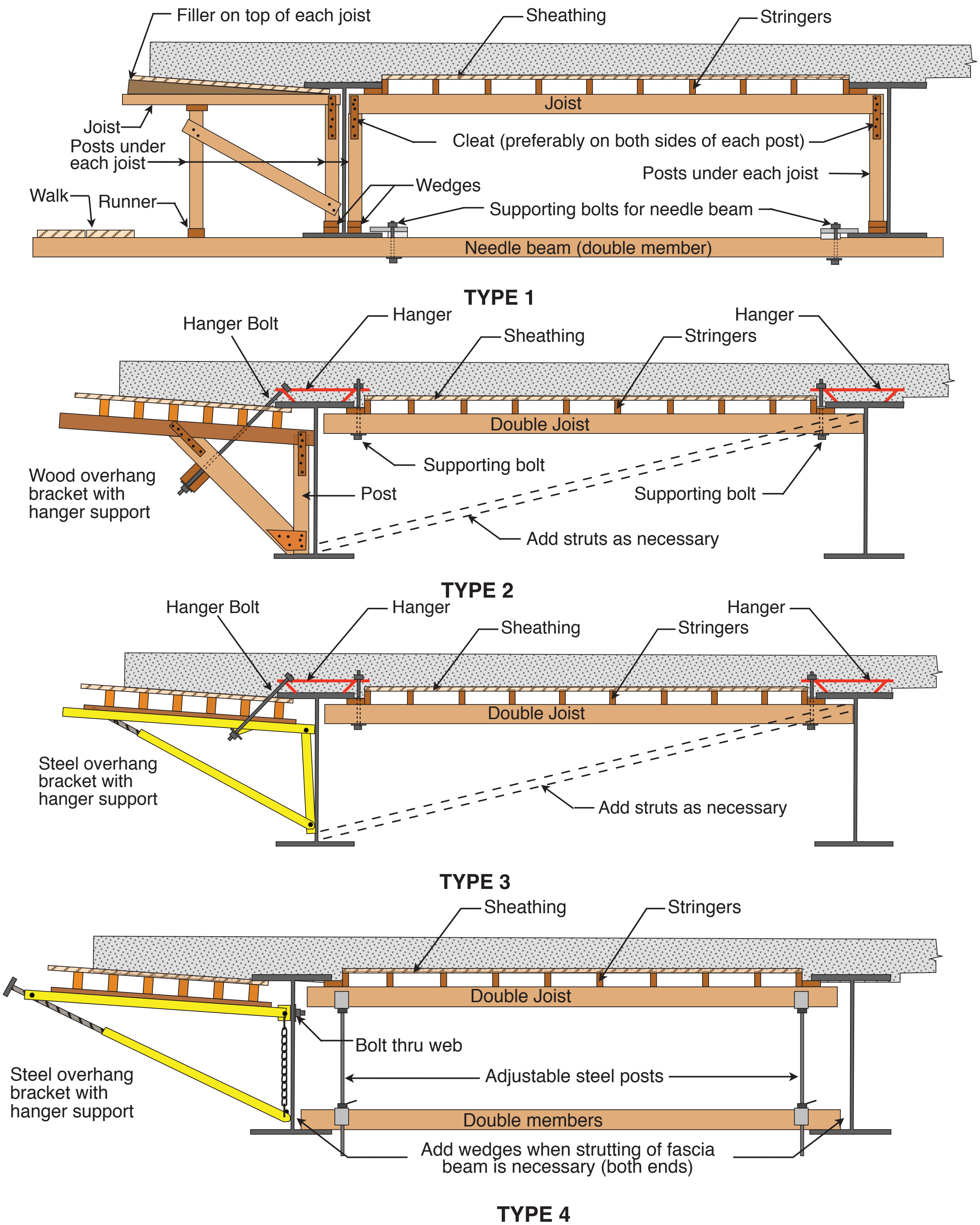
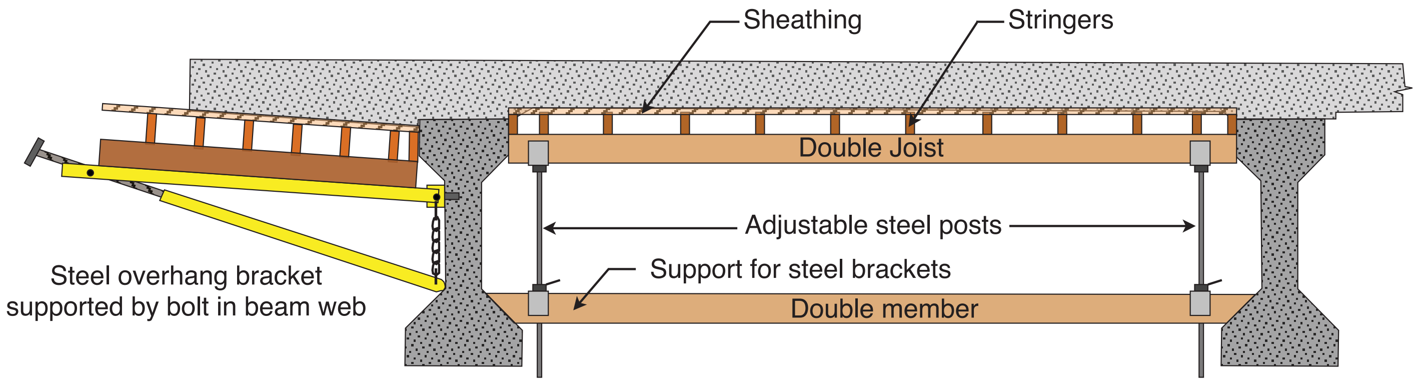
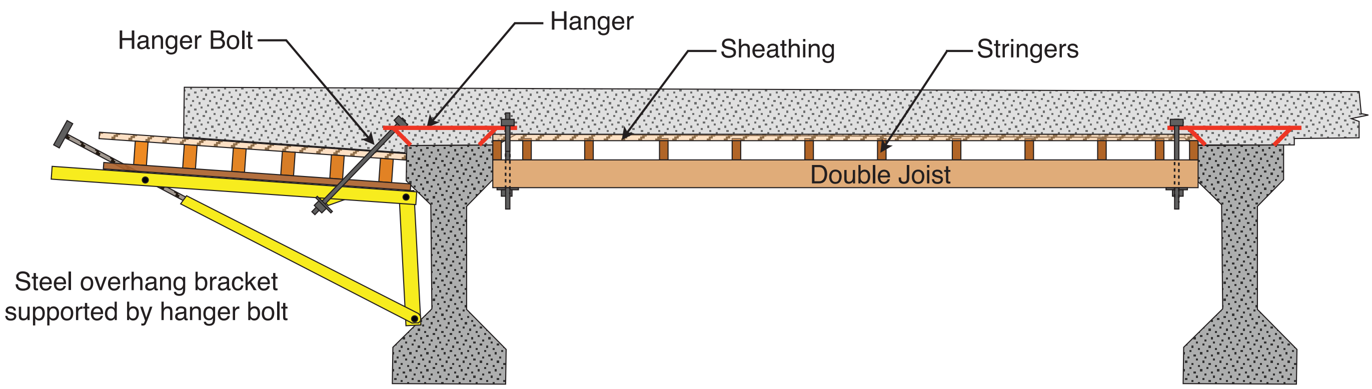


Figure 5-393-200-5– Details of several typical configurations used to support both roadway slab and overhang construction on bridges with steel beams.



TYPE 5



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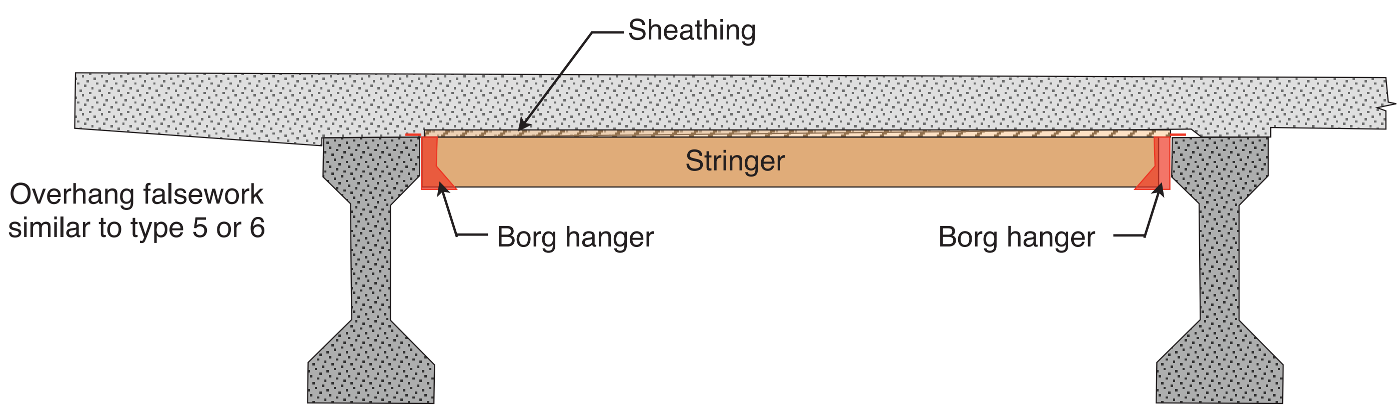


TYPE 7

Figure 5-393-200-6– Details of several typical configurations used to support roadway slab and overhang construction on bridges with prestressed concrete girders.

MAXIMUM ALLOWABLE PILE LOADS

Pile Dia, At Cut-off (inches)	Timber Piles (tons)	Steel Piles Friction (tons)	Steel Piles Point Bearing (tons)
	Butt diameters smaller than 8 in. are not permitted		
8	16	16	9,000 lb. per sq. inch of point area (or at least cross-sectional area of the pile)
10	20	20	
12	24	24	
14	28	28	
16	32		

Table 5-393-200-1– Maximum allowable loads in tons for driven piles used for falsework construction as a function of pile diameter.

upon drying the resulting lumber has the same dimensions as nominal, is said to be "full-sawn lumber." The dimensions of rough lumber can vary significantly and the Engineer must check the actual dimensions to see if they comply with the Contractor's forms and falsework plans. This is particularly true when laminated veneer lumber (LVL) or parallel strand lumber (PSL) is used, as this material is normally ripped out of wide billets and may not be sawn to dimensions that match the standard nominal or dressed lumber sizes.

In addition to these general requirements, it is specifically recommended that material for studs and walers be sized and dressed to at least S2E to provide for true concrete lines.

Lumber that must withstand stress should be checked for conformance with the appropriate allowable stresses shown in **Table 5-393-200-3** for allowable working stresses. The following notes apply to the use of information in this table:

1. **New Lumber:** Each piece of graded lumber is stamped with a grade stamp. On new material, information as to timber specie and stress grade can be obtained from this stamp for use with the allowable stress table in this section of the manual.

2. **Used Lumber:** In the event the grade stamp is missing or is unreadable, the species and grade or stress rating must be determined by visual examination or judgment or an assumed identification must be applied. In case of uncertainty, assume the lumber in question to be Red (Norway) Pine, and with a stress grade, such as No. 1 to be on the conservative side.

3. **Additional Considerations:** Regardless of whether new or used, a visual check should be made of stressed members with these considerations in mind:

    a. Any reduction in cross section at or near the middle 1/3 of the length of a beam reduces the capacity to resist bending. Such reduction in section could be a damaged area, large knots, notches, or holes in the lower 1/3 of the section. If such pieces are used for beams, only the sound portions of the section can be considered as effective for calculation of stresses.

    b. Notches or reduction in beam depth near the support point will reduce the beams capacity to resist horizontal shear stress. Special considerations and calculations are necessary to determine the horizontal shear stress when such pieces are used.

    c. When forms or falsework are constructed of previously used material that is judged to be not equal in strength to sound material, the allowable stresses in the table should be reduced by an appropriate amount.

Section Properties of Standard Lumber Sizes

NOMINAL SIZE 		DIMENSIONS OF DRESSED LUMBER S4S				MOMENT OF INERTIA $I = \frac{bd^3}{12}$ inches^4		SECTION MODULUS $S = \frac{bd^2}{6}$ inches^3	
b (in.)	d (in.)	b (in.)	d (in.)	Area b xd (in.^{2})	Weight (#/lin.ft.)	S4S (in.^{4})	Full Sawn (in.^{4})	S4S (in.^{3})	Full Sawn (in.^{3})
12	1	11 1/4	3/4	8.44	2.3	0.40	1.00	1.05	2.00
	1-1/4		1	11.25	3.1	0.94	1.95	1.88	3.13
	1-1/2		1-1/4	14.06	3.9	1.83	3.38	2.93	4.50
	2		1-1/2	16.88	4.7	3.16	8.00	4.22	8.00
2	4	1-1/2	3-1/2	5.25	1.5	5.36	10.67	3.06	5.33
	6		5-1/2	8.25	2.3	20.80	36.00	7.56	12.00
	8		7-1/4	10.88	3.0	47.63	85.33	13.14	21.33
	10		9-1/4	13.88	3.9	98.93	166.67	21.39	33.33
	12		11-1/4	16.88	4.7	177.98	288.00	31.64	48.00
	14		13-1/4	19.88	5.5	290.78	457.33	43.89	65.33
3	4	2-1/2	3-1/2	8.75	2.4	8.93	16.00	5.10	8.00
	6		5-1/2	13.75	3.8	34.66	54.00	12.60	18.00
	8		7-1/4	18.13	5.0	79.39	128.00	21.90	32.00
	10		9-1/4	23.13	6.4	164.89	250.00	35.65	50.00
	12		11-1/4	28.13	7.8	296.63	432.00	52.73	72.00
	14		13-1/4	33.13	9.2	484.62	686.00	73.15	98.00
4	4	3-1/5	3-1/2	12.25	3.4	12.51	21.33	7.15	10.67
	6		5-1/2	19.25	5.3	48.53	72.00	17.65	24.00
	8		7-1/4	25.38	7.0	111.15	170.67	30.66	42.67
	10		9-1/4	32.38	9.0	230.84	333.33	4.91	66.67
	12		11-1/4	39.38	10.9	415.28	576.00	73.83	96.00
	14		13-1/4	46.38	12.9	678.48	914.67	102.41	130.67
	16		15-1/4	53.38	14.8	1,034.42	1,365.33	135.66	170.67
6	6	5-1/2	5-1/2	30.25	8.4	76.26	108.00	27.73	36.00
	8		7-1/4	39.88	11.1	174.66	256.00	48.18	64.00
	10		9-1/4	50.88	14.1	362.75	500.00	78.43	100.00
	12		11-1/4	61.88	17.2	652.59	864.00	116.02	144.00
	14		13-1/4	72.88	20.2	1,066.18	1,372.00	160.93	196.00
	16		15-1/4	83.88	23.3	1,625.51	2,048.00	213.18	256.00
8	6	7-14	5-1/2	39.88	11.1	100.52	144.00	36.55	48.00
	8		7-1/4	52.56	14.6	230.22	341.33	63.51	85.33
	10		9-1/4	67.06	18.6	478.17	666.67	103.39	133.33
	12		11-1/4	81.56	22.7	860.23	1,152.00	152.93	192.00
	14		13-1/4	96.06	26.7	1,405.41	1,829.33	212.14	261.33
	16		15-1/4	110.56	30.7	2,142.72	2,730.67	281.01	341.33
10	6	9-1/4	5-1/2	50.88	14.1	128.25	180.00	46.64	60.00
	8		7-1/4	67.56	18.6	293.75	426.67	81.03	106.67
	10		9-1/4	85.56	23.8	610.08	833.33	131.91	166.67
	12		11-1/4	104.06	28.9	1,097.53	1,440.00	195.12	240.00
	14		13-1/4	133.56	34.0	1,793.11	2,286.67	270.66	326.67
	16		15-1/4	141.06	39.2	2,733.82	3,413.33	358.53	426.67
12	6	11-1/4	5-1/2	61.88	17.2	155.98	216.00	556.72	72.00
	8		7-1/4	81.56	22.7	357.26	512.00	98.55	128.00
	10		9-1/4	104.06	28.9	741.99	1,000.00	160.43	200.00
	12		11-1/4	126.56	35.2	1,334.84	1,728.00	237.3	288.00
	14		13-1/4	149.06	41.4	2,180.82	2,744.00	329.18	392.00
	16		15-1/4	171.56	47.7	3,324.92	4,096.00	436.05	512.00

Table 5-393-200-2– Section of properties of standard lumber sizes for both dressed lumber sizes and full sawn lumber sizes.

D. Allowable Stresses:

All of the engineering analysis of forms and falsework for work on projects under the *Standard Specifications for Construction* as published by the Minnesota Department of Transportation is based on the Allowable Stress Design (ASD) method. The allowable stresses and the modulus of elasticity (E) values listed in the tables contained herein are modified in accordance ACI Committee 347, *Guide to Formwork for Concrete* as published by the American Concrete Institute.

Some of the appropriate tabulate stresses (except E and compression perpendicular to grain) have been increased by 25% (C_D = 1.25) due to the anticipated short-term duration of the applied loads in most forms and falsework applications. See **Table 5-393-200-3** for allowable stresses for several of the common stress grades used for forms and falsework. Stresses for species or grades not listed in the accompanying tables should be obtained from the Office of Bridges and Structures at Mn/DOT and must conform to AASHTO Specifications.

The strength of wood column is dependent on several factors. The most significant factor is the column action: factor, C_P. The determination of the allowable working stress starts with the end bearing values and is modified by the ratio of the length divided by the least dimension of the column. This is called the "l over d Ratio" and is abbreviated (l/d). The (l/d) for a wood column must never exceed 50. The allowable compression parallel to grain (end bearing) in a wood column will be which ever is the least:

F'_c = the allowable end bearing, psi, listed in **Table 5-393-200-3,** page 5-393.200(11)

Or

$$F'_c = \frac{0.30E}{\left(\frac{l}{d}\right)^2}$$

where:

d = dimension of least side of column, in

l = unsupported length of column, in

E = modulus of elasticity, psi

The maximum allowable compressive stress for Douglas Fir columns and Red (Norway) Pine columns (as determined using the above criteria) may be obtained from **Chart 5-393-200-1.** Additionally, for convenience in making calculations involving dimension lumber, a tabulation of standard lumber sizes and their respective section properties are included in **Table 5-393-200-2**.

E. Plywood Sheathing:

General requirements for plywood sheathing are specified in the *Standard Specifications for Construction* as published by the Minnesota Department of Transportation. The plywood sheathing most commonly used is Douglas Fir Exterior "Plyform," which is available in three strength grades known as Class I, Class II, and Structural I. Plyform is a wood product like plywood that is made specially for concrete forms. All Classes are fabricated using exterior glue and have sanded grade B face plies. New panels of Plyform can be identified by the grade marks stamped on each panel as shown in **Figure 5-393-200-7,** page 5-393.200(13).

In considering the bending strength, rolling shear strength or deflection of a panel, only those plies that have their grain perpendicular to the supporting joist or studs are assumed to be resisting the load. The safe span length is therefore dependent not only on the Class of Plyform that is used but also on whether the grain of the face plies run across supports (perpendicular to the joist or studs), or parallel to supports (parallel to the joist or stud). When plywood or Plyform is used with the face grain running across the supports it is said that it is in the "strong direction" and when it is used with the grain of the face plies parallel to the supports it is used in the "weak direction." See **Figure 5-393-200-8** and **Figure 5-393-200-9** for details.

Section properties for Plyform Class I, Class II, and Structural I can be found in **Table 5-393-200-6.** The material properties for Plyform are shown in **Table 5-393-200-7.** The data in **Table 5-393-200-4** and **Table 5-393-200-5** and **Figure 5-393-200-15** may be used for quickly determining safe spacing of joist and studs when using Plyform Class I under two different loading conditions.

With the brand name or grade stamp on the plywood being used, the requirements of the *Standard Specifications for Construction* can be quickly verified. When no grade stamp is visible, it is the Contractor's responsibility to verify to the satisfaction of the Engineer that the concrete grade plywood has been furnished.

When it is determined that form grade plywood has been furnished but the specific Class of plywood is unknown, the following limiting stress values will apply:

ALLOWABLE WORKING STRESSES

Species and Commercial Grade	Size Restrictions	Bending Stress (psi)	Horizontal Shear (psi)	Side Bearing (psi)	End Bearing (psi)	Modulus of Elasticity, E (psi)
Douglas Fir-Larch	2 to 4" thick					
No. 1	< 10" width	1,375	220	625	1,875	1,700,000
No. 2	< 10" width	1,250	220	625	1,700	1,600,000
Southern Pine	2 to 4" thick					
No. 1	< 10" width	1,625	220	565	2,000	1,700,000
No. 2	< 10" width	1,300	220	565	1,875	1,600,000
Red Pine (Norway)	2" to 4" thick					
No. 1	< 10" width	1,065	175	335	1,250	1,100,000
Laminated Veneer Lumber (LVL)						
Douglas Fir - 2.0E		3,500	350	480	3,400	2,000,000
Southern Pine - 2.0E		3,650	350	525	3,800	2,000,000

NOTES:
The values in this table are based on Reference Design Values for the Species and Commercial Grades from the *National Design Specifications for Wood Construction*, 2005 Edition.
The bending and shear values have been adjusted using a Duration of Load Factor, C_D = 1.25.
All values have been adjusted for dry service conditions.
The Douglas Fir-Larch bending values have been adjusted by a Size Factor, C_F = 1.1.
Bending values have been adjusted by a Repetitive Member Factor, C_r = 1.00.

Table 5-393-200-3– Allowable working stresses for common species of wood used for construction of forms and falsework.

Maximum allowable bending stress, F_b : 1,500 psi

Maximum allowable shear stress, F_{rv} : 70 psi

Modulus of elasticity, E: 1,600,000 psi

Maximum compression perpendicular, F_c: 285 psi

Plywood section properties, which will be necessary for checking stresses when not using Plyform Charts that are tabulated in **Table 5-393-200-8,** page 5-393.200(16)**.**

The reuse of plywood sheathing will be dependent on its condition with respect to damage due to prior use, amount of permanent set from prior use, amount of face ply separation and the nature of the concrete surface being formed (exposed or not exposed, etc.). Plywood that is not longer suitable for its intended purpose must be rejected.

F. Structural Composite Lumber (SCL):

Structural Composite Lumber (SCL) is a term used to refer to three different wood-based manufactured structural products. The first type of SCL is Glued-laminated timber, generally referred to as “Glulam.” Glulam members are large structural members manufactured by gluing together 2 inch lumber using a exterior adhesives and pressure. Glulam members can be manufactured as either a beam element or column. They are identified by a unique Combination Symbol, which are similar to stress grades in lumber. The material properties for glulam member can be found in the *National Design Specifications for Wood Construction* (NDS). Glulam members have been in general use since

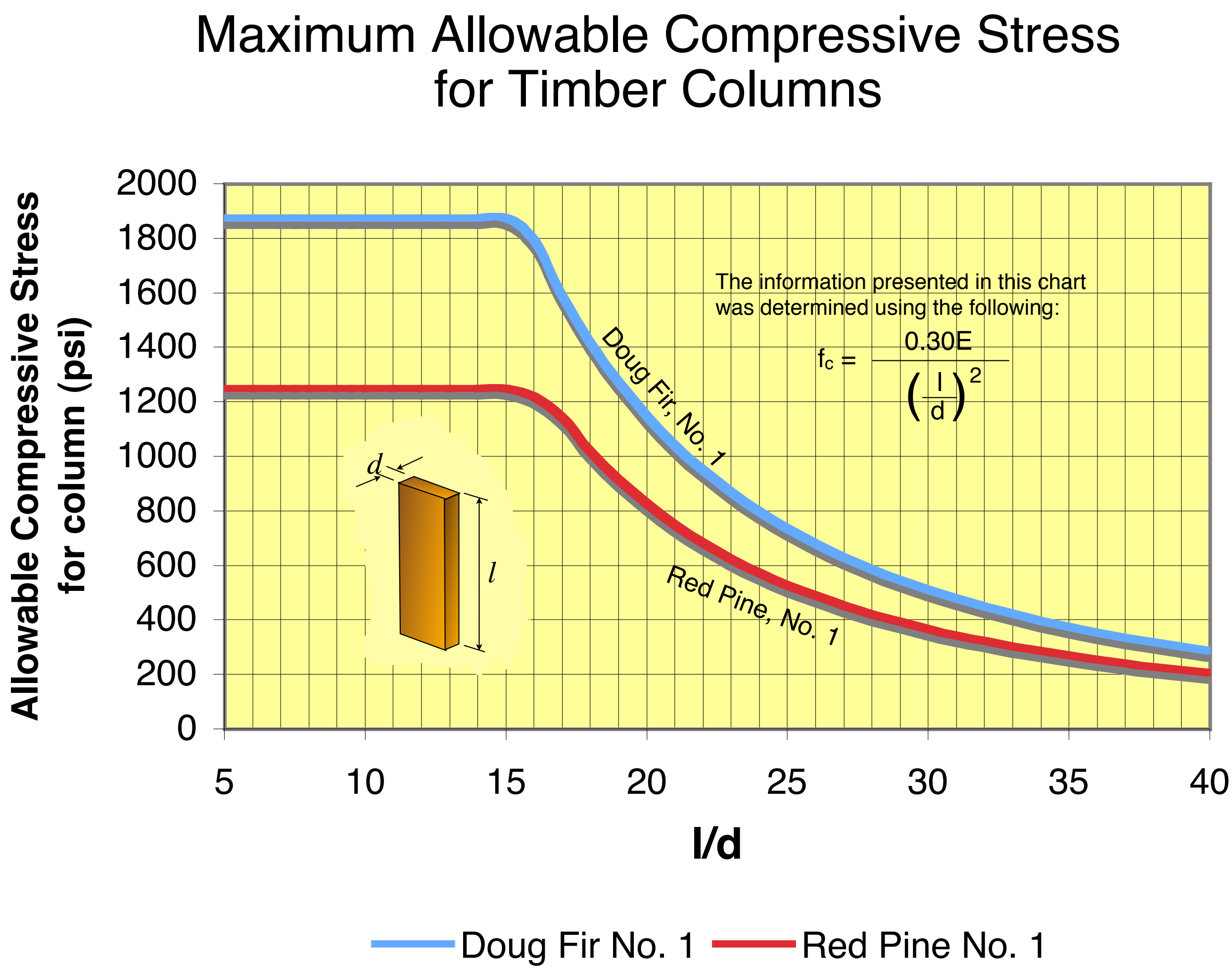


Chart 5-393-200-1– Maximum allowable compress stress parallel to grain for Douglas No. 1 and Red Pine, No. 1 used as columns.

1940's, but are used more frequently in permanent building construction.

Laminated Veneer Lumber (LVL) refers to wood structural members that have been manufactured by laminating thin layers of veneer (1/10 inch) using exterior adhesives under high clamping pressure and high temperature. These veneers are the same type of veneer plies that are used in the manufacture of plywood. However, in manufacture of LVL all of the plies run in the same direction, parallel to the longitudinal axis of the member. The edge view of LVL looks like plywood except all the plies run in the same direction. These elements are made in billets that are generally about 2 inches in thickness and with widths approximately 24 inches and lengths of 40 feet. These billets are generally "re-sawed" into smaller sizes similar to standard lumber.

There are several advantages to this material. First, the configuration of the veneers in the members create a timber member that has very low variability, this in turn provides high allowable strength properties. There are not generic tables of material properties for this type of material as each manufacturer of LVL has its own proprietary recipe for their material. However, all of the manufacturers print the significant material properties in big letters on every piece of their material.

The most significant material attribute for most timber structural elements is the modulus of elasticity, E. Most LVL have a large printed number such as 1.6 or 1.8 or 2.0. These numbers represent the modulus of elasticity in millions. Where 1.8 means the member has an *E* of 1,800,000 psi, and a piece of LVL with 2.0 printed on it has an *E* of 2,000,000 psi. The LVL members also have another number printed in smaller letters that state the allowable bending stress. There will be printing on the

LVL that looks like "Fb1600", which means the allowable bending stress is 1,600 psi. Most of the strength attributes of wood structural members are closely correlated to the modulus of elasticity. Most strength attributes and use guidelines are readily available in the manufacturers published literature.

The next type of SCL is Parallel Strand Lumber (PSL). One type of parallel strand lumber is made from the same species of wood used for plywood (i.e., Douglas Fir and Southern Pine). It starts with a sheet of veneer that is then clipped into narrow strands that are approximately ½ inch in width and 8 feet in length. The strands are dried, coated with a waterproof adhesive, and bonded together under pressure and heat. Like other SCL products the major material properties are printed on the members. All strength properties and use guidelines are available in the manufacturers published literature.

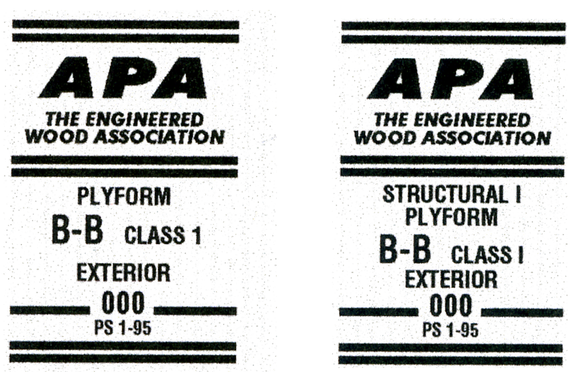


Figure 5-393-200-7– Examples of grade stamps commonly used on Plyform material used for forms and falsework.

G. Form Lining:

General requirements for Form Liners, both as to material and usage, are specified in detail in the *Standard Specifications for Construction* as published by the Minnesota Department of Transportation. Forms incorporating a form liner backed by sheathing will be used rarely except in the case of architectural treatment of concrete surfaces. Projects that require form linings will have the requirements contained in the contract documents.

H. Rolled Steel Shapes:

When angles, channels, wide flange beams, H-piles or other rolled steel shapes are used in critical portions of the falsework, the section should be identified by making complete measurements of the cross section. These dimensions can then be used to identify the member further by referring to the *AISC Steel Construction Manual*, where all standard rolled sections are listed along with their dimensions, unit weights, and all the necessary design properties. Since this material cannot be visually identified as to grade of steel, the following stress limits should be assumed, unless the Contractor furnishes satisfactory assurance that the steel is of a higher grade.

Rolled Steel Shapes (assume ASTM A36 steel)

Allowable bending stress, F_b': 25,000 psi

Allowable compressive stress, F_c':

$$F'_c = 16{,}980 - 0.53 \times \left(\frac{KL}{r}\right)^2$$

where:

L = unsupported length, in

K = 1.0 for pined ends (ref: AISC Manual)

r = governing radius gyration, in

and:

L/r must not exceed 120

The values listed above will be sufficient for checking most falsework problems involving rolled steel members. Any additional design considerations (such as steel falsework and other special cases) should conform to the provisions of the AASHTO Standard Specifications for Highway Bridges, as required in the *Standard Specifications for Construction* as published by the Minnesota Department of Transportation

When previously used material is to be incorporated into the work, the extent of damage (caused by prior usage) and corrosion should be evaluated. If corrosion is determined to have reduced the net thickness of the cross section, it is allowable to use the section properties of a similar rolled shape in the AISC manual with thickness dimensions compared to those of the net intact material. Additional requirements for use of structural shapes are given in the *Standard Specifications for Construction* as published by the Minnesota Department of Transportation.

I. Proprietary Products:

The increasing use of special devices, (made of material other than wood) for forms and falsework has, in general, resulted is a speed-up of work as well as improved quality of work. However, there is usually a degree of uncertainty about each new device until it is proven in use. A partial listing of devices which have been used successfully, and in some instances unsuccessfully, is as follows:

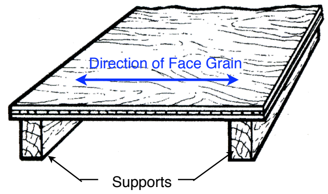


Figure 5-393-200-8– Illustrates plywood or plywood that is used in the strong direction where the face grain is perpendicular to the supports.

RECOMMENDED MAXIMUM PRESSURES ON PLYFORM CLASS I (psf)[(a)(c)]
FACE GRAIN ACROSS SUPPORTS[(b)]

Support Spacing (in.)	Plywood Thickness (in.)													
	15/32		1/2		19/32		5/8		23/32		3/4		1-1/8	
4	2715	2715	2945	2945	3110	3110	3270	3270	4010	4010	4110	4110	5965	5965
8	885	885	970	970	1195	1195	1260	1260	1540	1540	1580	1580	2295	2295
12	355	395	405	430	540	540	575	575	695	695	730	730	1370	1370
16	150	200	175	230	245	305	265	325	345	390	370	410	740	770
20	-	115	100	135	145	190	160	210	210	270	225	285	485	535
24	-	-	-	-	-	100	-	110	110	145	120	160	275	340
32	-	-	-	-	-	-	-	-	-	-	-	-	130	170

(a) Deflection limited to 1/360th of the span, 1/270th where shaded.
(b) Plywood continuous across two or more spans.
(c) ACI recommends a minimum lateral design pressure of 600 C_W but it need not exceed p = wh.
Source: APA — *The Engineered Wood Association*

Table 5-393-200-4– Maximum allowable pressure for Plyform Class I that is used for the design of forms and falsework when used in the strong direction.

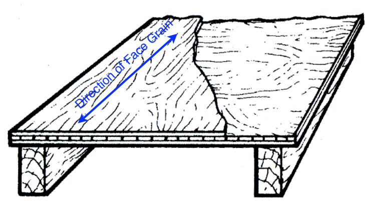


Figure 5-393-200-9– Illustrates plywood or plywood that is used in the weak direction where the face grain is parallel to the supports.

RECOMMENDED MAXIMUM PRESSURES ON PLYFORM CLASS I (psf)[(a)(c)]
FACE GRAIN PARALLEL SUPPORTS[(b)]

Support Spacing (in.)	Plywood Thickness (in.)													
	15/32		1/2		19/32		5/8		23/32		3/4		1-1/8	
4	1385	1385	1565	1565	1620	1620	1770	1770	2170	2170	2325	2325	4815	4815
8	390	390	470	470	530	530	635	635	835	835	895	895	1850	1850
12	110	150	145	195	165	225	210	280	375	400	460	490	1145	1145
16	-	-	-	-	-	-	-	120	160	215	200	270	710	725
20	-	-	-	-	-	-	-	-	115	125	145	155	400	400
24	-	-	-	-	-	-	-	-	-	-	-	100	255	255

(a) Deflection limited to 1/360th of the span, 1/270th where shaded.
(b) Plywood continuous across two or more spans.
(c) ACI recommends a minimum lateral design pressure of 600 C_W but it need not exceed p = wh.
Source: APA —*The Engineered Wood Association*

Table 5-393-200-5– Maximum allowable pressure for Plyform Class I that is used for the design of forms and falsework when used in the weak direction.

SECTION PROPERTIES FOR PLYFORM CLASS I AND CLASS II, AND STRUCTURAL I PLYFORM

		Properties for Stress Applied Parallel with Face Grain			Properties for Stress Applied Perpendicular to Face Grain		
Thickness (inches)	Approx. Weight (psf)	Moment of Inertia I ($in.^4/ft$)	Effective Section Modulus KS ($in.^3/ft$	Rolling Shear Constant lb/Q ($in.^2/ft$)	Moment of Inertia I $in.^4/ft$	Effective Section Modulus KS ($in.^3/ft$)	Rolling Shear Constant lb/Q (in^2/ft)
CLASS I							
15/32	1.4	0.066	0.244	4.743	0.018	0.107	2.419
1/2	1.5	0.077	0.268	5.133	0.024	0.130	2.739
19/32	1.7	0.115	0.335	5.438	0.029	0.146	2.834
5/8	1.8	0.130	0.358	5.717	0.038	0.175	3.094
23/32	2.1	0.180	0.430	7.009	0.072	0.247	3.798
3/4	2.2	0.199	0.455	7.187	0.092	0.306	4.063
7/8	2.6	0.296	0.584	8.555	0.151	0.422	6.028
1	3.0	0.427	0.737	9.374	0.270	0.634	7.014
1-1/8	3.3	0.554	0.849	10.43	0.398	0.799	8.419
CLASS II							
15/32	1.4	0.063	0.243	4.499	0.015	0.138	2.434
1/2	1.5	0.075	0.267	4.891	0.020	0.167	2.727
19/32	1.7	0.115	0.334	5.326	0.025	0.188	2.812
5/8	1.8	0.130	0.357	5.593	0.032	0.225	3.074
23/32	2.1	0.180	0.430	6.504	0.060	0.317	3.781
3/4	2.2	0.198	0.454	6.631	0.075	0.392	4.049
7/8	2.6	0.300	0.591	7.99	0.123	0.542	5.997
1	3.0	0.421	0.754	8.614	0.220	0.812	6.987
1-1/8	3.3	0.566	0.869	9.571	0.323	1.023	8.388
STRUCTRAL I							
15/32	1.4	0.067	0.246	4.503	0.021	0.147	2.405
1/2	1.5	0.078	0.271	4.908	0.029	0.178	2.725
3/5	1.7	0.116	0.338	5.018	0.034	0.199	2.811
5/8	1.8	0.131	0.361	5.258	0.045	0.238	3.073
23/32	2.1	0.183	0.439	6.109	0.085	0.338	3.78
3/4	2.2	0.202	0.464	6.189	0.108	0.418	4.047
7/8	2.6	0.317	0.626	7.539	0.179	0.579	5.991
1	3.0	0.479	0.827	7.978	0.321	0.870	6.981
1-1/8	3.3	0.623	0.955	8.841	0.474	1.098	8.377

Source: APA — *The Engineered Wood Association*

Table 5-393-200-6– Section properties for Plyform Class I, Class II, and Structural I based on the direction of the face grain relative to the direction of the supports.

MATERIAL PROPERTIES FOR PLYFORM

	Plyform Class I	Plyform Class II	Structural I Plyform
Modulus of elasticity — E (psi, adjusted, use for bending deflection calculation)	1,650,000	1,430,000	1,650,000
Modulus of elasticity — E_e (psi, adjusted, use for shear deflection calculations)	1,500,000	1,300,000	1,500,000
Bending — F_b (psi)	1,930	1,330	1,930
Rolling shear stress — F_s (psi)	72	72	102

Source: APA-*The Engineered Wood Association*

Table 5-393-200-7– Material properties of Plyform Class I, Class II, and Structural I.

SECTION PROPERTIES OF PLYWOOD

Sanded Plywood Net Thickness (in.)	Number of Plies	Effective Thickness for Shear All Grades, Using Exteriior Glue (in.)	12-in. wide, used with face grain perpendicular to supports				12-in. wide, used with face grain perpendicular to supports			
			Area for Tension and Compression (in.^{2})	Moment of Inertia I (in.^{4})	Effective Section Modulus S (in.^{3})	Rolling Shear Constant I/Q (in.)	Area for Tension and Compression (in.^{2})	Moment of Inertia I (in.^{4})	Effective Section Modulus S (in.^{3})	Rolling Shear Constant I/Q (in.)
1/4	3	0.241	1.680	0.013	0.091	0.179	0.600	0.001	0.016	-
3/8	3	0.305	1.680	0.040	0.181	0.309	1.050	0.004	0.044	-
1/2	5	0.450	2.400	0.080	0.271	0.436	1.200	0.016	0.096	0.215
5/8	5	0.508	2.407	0.133	0.360	0.557	1.457	0.040	0.178	0.315
3/4	5	0.567	2.778	0.201	0.456	0.687	2.200	0.088	0.305	0.393
7/8	7	0.711	2.837	0.301	0.585	0.704	2.893	0.145	0.413	0.531
1	7	0.769	3.600	0.431	0.733	0.763	3.323	0.234	0.568	0.632
1 1/8	7	0.825	3.829	0.566	0.855	0.849	3.307	0.334	0.702	0.748

Table 5-393-200-8– Section properties for Plywood based on the direction of the face grain relative to the direction of the supports.

MAXIMUM SPANS FOR LUMBER FRAMING, INCHES — DOUGLAS-FIR NO. 2 OR SOUTHERN PINE NO. 2

Equivalent Uniform Load (lb/ft)	Continuous Over 2 or 3 Supports (1 or 2 Spans) Nominal Size							Continuous Over 4 or More Supports (3 or 4 Spans) Nominal Size						
	2x4	2x6	2x8	2x10	4x4	4x6	4x8	2x4	2x6	2x8	2x10	4x4	4x6	4x8
200	48	73	92	113	64	97	120	56	81	103	126	78	114	140
400	35	52	65	80	50	79	101	39	58	73	89	60	88	116
600	29	42	53	65	44	64	85	32	47	60	73	49	72	95
800	25	36	46	56	38	56	73	26	41	52	63	43	62	82
1000	22	33	41	50	34	50	66	22	35	46	56	38	56	73
1200	19	30	38	46	31	45	60	20	31	41	51	35	51	67
1400	18	28	35	43	29	42	55	18	28	37	47	32	47	62
1600	16	25	33	40	27	39	52	17	26	34	44	29	44	58
1800	15	24	31	38	25	37	49	16	24	32	41	27	41	55
2000	14	23	29	36	24	35	46	15	23	30	39	25	39	52
2200	14	22	28	34	23	34	44	14	22	29	37	23	37	48
2400	13	21	27	33	21	32	42	13	21	28	35	22	34	45
2600	13	20	26	31	20	31	41	13	20	27	34	21	33	43
2800	12	19	25	30	19	30	39	12	20	26	33	20	31	41
3000	12	19	24	29	18	29	38	12	19	25	32	19	30	39
3200	12	18	23	28	18	28	37	12	19	24	31	18	29	38
3400	11	18	22	27	17	27	35	12	18	24	30	18	28	36
3600	11	17	22	27	17	26	34	11	18	23	30	17	27	35
3800	11	17	21	26	16	25	33	11	17	23	29	16	26	34
4000	11	16	21	25	16	24	32	11	17	22	28	16	25	33
4200	11	16	20	25	15	24	31	11	17	22	28	16	24	32
4400	10	16	20	24	15	23	31	10	16	22	27	15	24	31
4600	10	15	19	24	14	23	30	10	16	21	26	15	23	31
4800	10	15	19	23	14	22	29	10	16	21	26	14	23	30
5000	10	15	18	23	14	22	29	10	16	21	25	14	22	29

NOTE: Spans are based on the 2001 NDS allowable stress values. Where: C_D = 1.25, C_r = 1.0. C_M = 1.0
Deflection is limited to 1/360th of span with 1/4" maximum
Spans within the brown shaded boxes are controlled by deflection. Bending governs elsewhere.

Table 5-393-200-9– Maximum allowable span lengths for Douglas Fir, No. 2 and Southern Pine, No. 2 framing lumber based on varying uniform loads.

MAXIMUM SPANS FOR LUMBER FRAMING, INCHES — HEM-FIR NO. 2

Equivalent Uniform Load	Continuous Over 2 or 3 Supports (1 or 2 Spans) Nominal Size							Continuous Over 4 or More Supports (3 or 4 Spans) Nominal Size						
(lb/ft)	2x4	2x6	2x8	2x10	4x4	4x6	4x8	2x4	2x6	2x8	2x10	4x4	4x6	4x8
200	45	70	90	110	59	92	114	54	79	100	122	73	108	133
400	34	50	63	77	47	74	96	38	56	71	87	58	86	112
600	28	41	52	63	41	62	82	29	45	58	71	48	70	92
800	23	35	45	55	37	54	71	23	37	48	61	41	60	80
1000	20	31	40	49	33	48	64	20	32	42	53	37	54	71
1200	18	28	36	45	30	44	58	18	28	37	47	33	49	65
1400	16	25	33	41	28	41	54	16	26	34	43	29	45	60
1600	15	23	31	39	25	38	50	15	24	31	40	26	41	54
1800	14	22	29	37	23	36	48	14	22	30	38	24	38	50
2000	13	21	28	35	22	34	45	14	21	28	36	22	35	46
2200	13	20	26	33	20	32	42	13	20	27	34	21	33	43
2400	12	19	25	32	19	30	40	12	20	26	33	20	31	41
2600	12	19	25	30	18	29	38	12	19	25	32	19	30	39
2800	12	18	24	29	18	28	36	12	18	24	31	18	28	37
3000	11	18	23	28	17	26	35	11	18	24	30	17	27	36
3200	11	17	22	27	16	25	34	11	17	23	29	17	26	34
3400	11	17	22	27	16	25	32	11	17	22	29	16	25	33
3600	11	17	21	26	15	24	31	11	17	22	28	16	24	32
3800	10	16	21	25	15	23	31	10	16	22	28	15	24	31
4000	10	16	20	24	14	23	30	10	16	21	27	15	23	30
4200	10	15	20	24	14	22	29	10	16	21	27	14	22	30
4400	10	15	19	23	14	22	28	10	16	21	26	14	22	29
4600	10	15	19	23	13	21	28	10	15	20	26	14	21	28
4800	10	14	18	22	13	21	27	10	15	20	25	13	21	28
5000	10	14	18	22	13	20	27	10	15	20	24	13	21	27

NOTE: Spans are based on the 2001 NDS allowable stress values. Where: C_D = 1.25, C_r = 1.0. C_M = 1.0
Deflection is limited to 1/360th of span with 1/4" maximum
Spans within the brown shaded boxes are controlled by deflection. Bending governs elsewhere.

Table 5-393-200-10– Maximum allowable span lengths for Hem-Fir, No. 2 framing lumber based on varying uniform loads.

1. **Wall Form Panels:** The form panels referred to herein are mass-produced brand name form sections (constructed either of steel or steel and wood) which are produced in small segments so as to be adaptable to a variety of concrete shapes and a variety of types of construction. Past experience with certain brands of these form panels resulted in the recommendation that form panels construction should not be permitted for concrete exposed to view. The reason for dissatisfaction on the work referred to was as follows:

   a. Objectionable offsets existed at abutting panels edges.

   b. There were an excessive number of joints. (The frequency of panel joints should generally be no greater than in conventional plywood-form construction.)

   c. After being reused a number of times, permanent set (permanent deflection) in panels became excessive.

   d. Adequate provisions were not provided for the overall alignment of the formwork, nor for creating mortar-tight joints in the completed form.

Only a form panel system that overcomes these objections, with respect to appearances, can be considered for use on concrete surfaces exposed to view.

The design of the forms, with respect to size and spacing of members, is normally furnished by the manufacturer, either as part of the advertising literature or as a special design for the job along with a safe rate of pour for concrete in the form system. These should be carefully adhered to.

2. Circular Column Forms: Specific requirements for circular column forms are stated in the *Standard Specification for Construction*. Such forms have been fabricated of steel, fiberglass, and paper or other fibers, and all have been used with varying degrees of success.

Since some circular forms can be damaged through mishandling or improper storage, it is necessary to check the roundness and smoothness when making a judgment as to acceptability of each form. The form diameter or any axis should not be more than ½ inch less than the specified diameter. This requirement is to assure proper cover of column reinforcement. The roundness of paper tubes is normally not so critical since concrete pressure during filling will round out the tube.

Reusable steel forms are susceptible to damage in the form of small dents and kinks. These result in unsightly dimples on the concrete surface. Such forms should normally be required prior to permitting their use. In addition, abutting panels should be adjusted to eliminate offsets at panel joints.

Due to the possibility of very fast rates of concrete placement in column forms, the pressure at the bottom of the form can be extremely high. Fasteners for the vertical form-joint on segmental forms (such as steel or fiberglass column forms) can readily be checked for ability to withstand these pressures. These forms usually have provisions for a variable number of bolts or pins in these joints to allow a wide range of loading conditions.

Since circular paper or fiber forms are commercially mass-produced in several different strength grades, the adequacy of their design for a specific case will normally be determined by checking the manufacturer's literature. Note carefully whether this literature contains a safe loading, failure loading or bursting pressure. When only the bursting pressure is given, a safety factor must be applied to determine a safe load. Normally a safety factor of 2 is adequate.

If paper tubes have become wet prior to use, they should be inspected for weak areas in advance of concrete placement. Paper should also be checked to assure that no conspicuous seam ridges are present on the inside surfaces since these cause objectionable spiral ridges on the finished concrete surface.

3. **Friction Collars for Pier Caps:** Friction collars are steel devices that are clamped around to the top of circular concrete columns to support the pier cap falsework and pier cap concrete. Serious failures have resulted because of inattention to the placement of these collars. Since the entire falsework, in this case, is dependent on the stability of the collar, the tightening of the collar must be properly performed. These collars must be level to assure bearing on the concrete. See **Photograph 5-393-200-1** for a close-up of a typical friction collar used to support the forms and falsework for pier construction. The manufacturer's literature should be used to determine the minimum necessary bolt tension. In addition, the total applied vertical load must not exceed the safe load specified in the manufacturer's literature.

Photograph 5-393-200-1– A view of friction collar used to support the falsework for the construction of a concrete pier cap.

Another type of product used to support falsework for this type of construction is similar to a friction collar except that it uses a steel bar placed into a steel pipe that was cast into the pier column. The entire vertical load on the collar is transferred to the pier column via a steel bar inside the steel pipe. The analysis of this type of construction should include the capacity of the steel bar and the bearing capacity of the embedded pipe in the pier column.

4. Slab Falsework—Interior Bays: Several types of slab falsework other than the all-wood designs, which have been successfully used by Contractors are as follows:

a. **Adjustable Steel Post:** This system basically replaces the wood legs of the wooden "horse" system with adjustable steel posts. These posts are normally supported on wood joists spanning between the bottom flanges of adjacent beams. The strength of this type of system will normally be controlled by the strength of the wood members used in the system. An example of this type of falsework can be see in TYPE 5 of **Figure 5-393-200-6**.

b. **Steel Hangers:** This is basically a hardware item that is laid transversely across the top flange of the beam to receive a vertical bolt on either side of the flange. The bolt in turn supports the main falsework member (joist). Safe working loads for steel hangers are listed in the manufacturers literature. See **Figure 5-393-200-5** and **Figure 5-393-200-6** for details. One other type of hardware used as joist hanger can be seen in **Photograph 5-393-200-2**.

c. **Steel Bar Joists:** This is a type of steel member for falsework that can be adjusted to a variety of lengths. Load capacity, allowable spacing and deflection data are available from the manufacturer's literature and should be used for checking the system. Such steel bar joists have been used as joists to support longitudinal falsework stringers, and are also used at closer spacing with sheathing placed directly on them. In the event the latter system is used, and no wood nailer is available to hold down the sheathing then system of wire ties or some other approved method of hold down will be necessary. Precautions must be taken to allow for residual camber in this type of falsework system. The amount of residual camber anticipated after placement of the concrete should be determined (by field tests if necessary) and adequate allowance made in setting stool heights to obtain the specified slab thickness.

d. **Corrugated Steel Forms:** These are commercially mass-produced corrugated sheet metal forms for the bottom of the slab that require no additional supporting falsework. Each unit spans transversely from beam to beam on the bridge and acts in the capacity of a complete structural entity of falsework and sheathing. These units are galvanized and are normally intended to remain in place at completion of the work. Safe loads and deflections for each size of member are available in the manufacturer's literature.

Photograph 5-393-200-2– A close-up of a typical steel joist hanger used as a bracket to support the ends on joists used for falsework for the interior bays roadway slab construction.

5. Slab Overhang Falsework: Several types of slab falsework other than the all-wood designs, which have been successfully used by Contractors are as follows:

a. **Steel Overhang Brackets:** Typical applications of steel overhang brackets are shown **Figure 5-393-200-5** and **Figure 5-393-200-6.** See **Photograph 5-393-200-3** for typical applications of steel overhang brackets. Details and design data pertaining to two commonly used overhang brackets (Capitol and Superior) are found in **Figure 5-393-200-10** and **Figure**

5-393-200-11 and **Figure 5-393-200-12.** It is intended that spacing and deflection of these brackets be determined by these details as furnished by the manufacturers. However, several precautions must be observed as described below. Information for the Capitol brackets states that the brackets should be spaced at 6' – 0" centers. However, experience has shown that the 6' – 0" spacing must be reduced under certain conditions. For example, when the strike-off rails are placed on the top of the coping forms or when a very wide slab overhang is specified in the plans, a much higher load is applied to each bracket unless this spacing is reduced.

Photograph 5-393-200-3– A close-up of over hang brackets used to support the concrete overhang. Shown used on a bridge with steel beams.

When installing Capitol brackets, the 2" x 4" member placed in the top horizontal member of the bracket must be firmly seated and the hanger "chain" must be tight. Poorly aligned concrete surface have resulted when seating occurred during concrete placement.

The influence lines shown in **Figure 5-393-200-12,** page 5-393.200(23) for checking the Superior brackets may be used with a variety of loading conditions. The actual load in the critical members can be determined by use of this chart and checked against the safe working loads shown in the Figure.

A wood filler block is required when using these brackets on prestressed concrete girders, see **Figure 5-393-200-11.**. This filler must be fitted as necessary to provide a flat bearing surface on the beam at the end of the top horizontal members and at the end of the diagonal member. The filler must not bear on the vertical member of the Superior bracket.

The deflection graphs given for each of these brackets should be used only as a guide, since the graph applies only to the specific loading pictured on the manufacturer's details. When unusual loading conditions are encountered a full-scale field test is recommended for either bracket. An over-load should be applied to assure that there is a safety factor.

Since cantilever brackets tend to rotate the fascia beam (push the bottom flange inward), special bracing precautions, as specified in the *Standard Specifications*, are occasionally necessary. This is particularly true when the length of the overhang is greater than the depth of the fascia beam.

For beams depths of 24" or less, the difficulty of obtaining good concrete lines increases when this type of overhang falsework is used, and serious consideration should be given to the use of the needle beams as shown as TYPE 1 on **Figure 5-393-200-5.**

b. **Steel Hangers:** These were described in 4 (b) above.

6. **Tubular Steel Scaffolding:** The basic components of this type of scaffold shoring are end frames of various designs and dimensions that are assembled with diagonal bracing and lock clamps. Vertical adjustments are made by adjustable jacks either at the bottom or top of the frames. Frames are normally fitted with flat top plates or U-shaped heads for supporting the falsework and forms. See **Photograph 5-393-200-4** for a view of typical application of tubular steel scaffolding used as falsework for concrete slab-span construction.

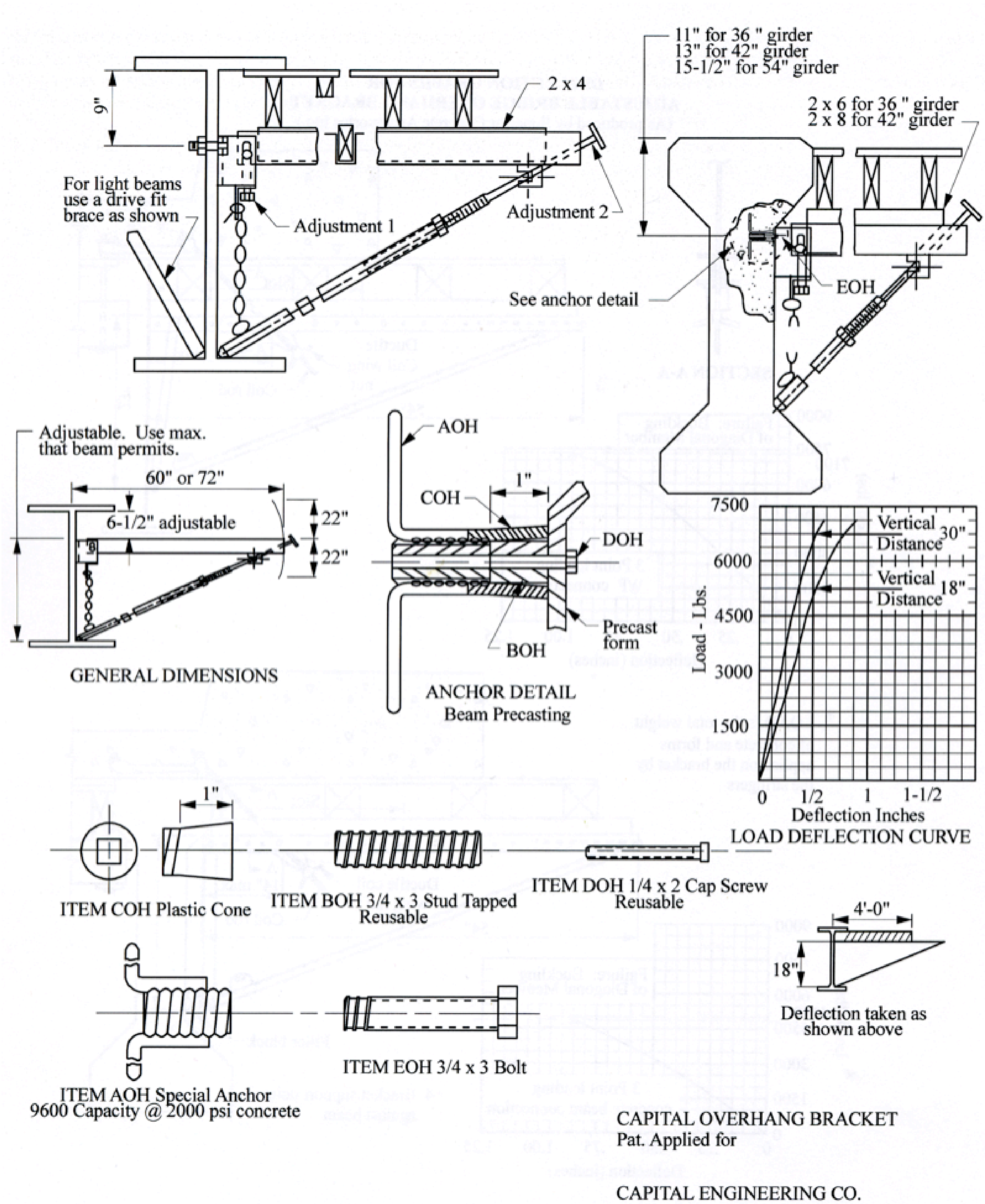


Figure 5-393-200-10– Engineering information and design details for the use Capital Overhang Brackets manufactured y Capital Engineering Co.

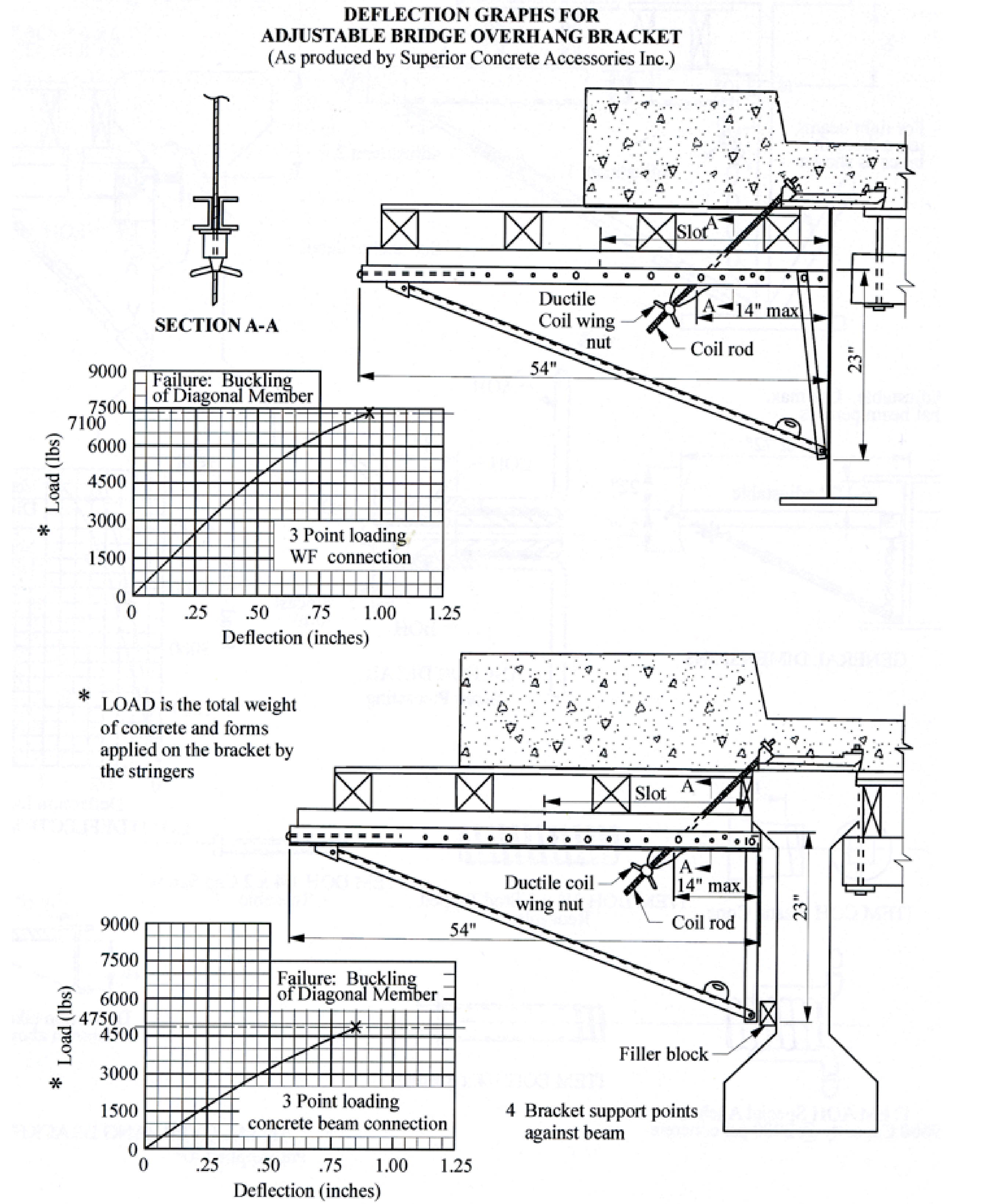


Figure 5-393-200-11– Engineering information and design details for the Superior Overhang Bracket manufactured by Superior Concrete Accessories, Inc.

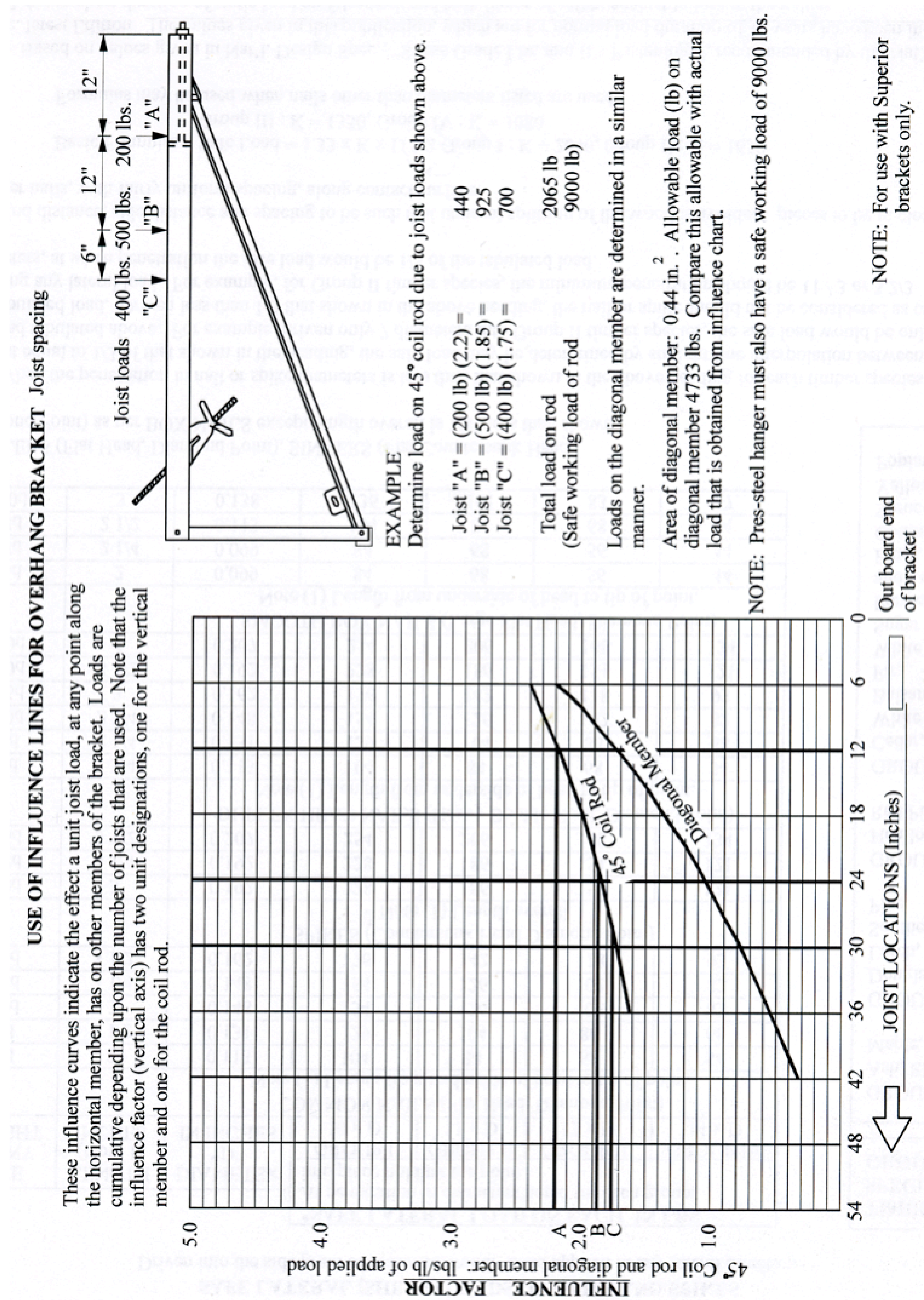


Figure 5-393-200-12– Engineering information and design details for the Superior Overhang Bracket manufactured by Superior Concrete Accessories, Inc.

Photograph 5-393-200-4– A view of the use of tubular steel scaffolding used as falsework for the construction of a concrete slab-span bridge.

These towers are rated by the load carrying capacity of either one leg or one frame (two legs). The manufacturer's rated capacity should not be exceeded. Adequate rigid bracing involving several units of steel shoring should be provided. Full bearing for the base plates should be provided, such as being set in fresh mortar when resting on rock-like formations. Mudsills placed on yielding earth are not permitted for supports.

7. Void Tubes for Voided Slabs: These tubes are similar to the tubes used for column forms except that galvanized steel tubes are also permitted. The circumferential crushing pressure and straight crushing pressure of these tubes will normally be listed in the manufacturer's literature. When checking stresses, it is necessary to determine if the manufacturer has listed a safe pressure or a failure pressure.

Since stress in the void tubes is very high at the tie-down points, a careful visual inspection is necessary at those locations. Wetting of paper tubes can result in isolated weak spots where the waterproof coating has been scratched or damaged and water has penetrated into the paper or fiber layers. Such pieces should be rejected unless they can be satisfactorily reinforced.

Void tubes must be mortar tight. When several lengths of tubes are necessary to make up the length of void shown in the Plan, each segment of tube should have sealed ends. Butting tube ends together and taping around the perimeter of the joint is normally unacceptable since deformation of one of the jointed tubes during concrete placement would likely rupture a tape splice.

8. Nails: Nails are the most commonly used connectors for wood forms and falsework. The resistance to lateral loads on nails and spikes is directly correlated to the diameter of the nail or spike. See **Figure 5-393-200-14** for a diagram of the size of nails and spikes. **Table 5-393-200-11** contains the allowable lateral loads on various sizes of nails and spikes when used in several different species of wood. Many of the newer power nailing devices use nails that do not necessarily match the diameters of the traditional nail sizes as identified by the "penny weight", denoted by a small letter d. The actual diameter of the nails used should be measured with a caliper.

9. **Form Bolts:** General requirements governing bolts or form tie are given in the relevant provisions contained in the *Standard Specifications for Construction* as published by the Minnesota Department of Transportation. One specific provision is that a major portion of the tie device must remain permanently in the concrete. The device must also be designed so that all material in the device to a depth of 1 inch can be disengaged and removed without spalling or damaging the concrete. Several types of commercially available form ties meeting this description are shown in **Figure 5-393-200-13**.

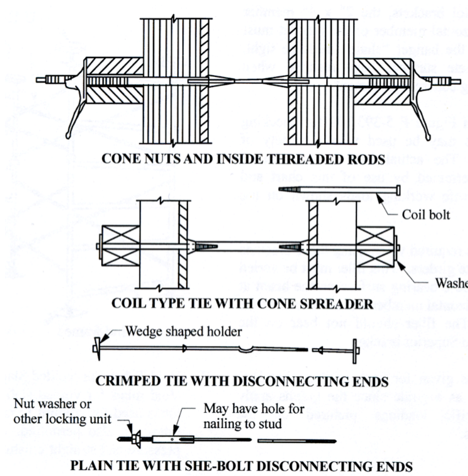
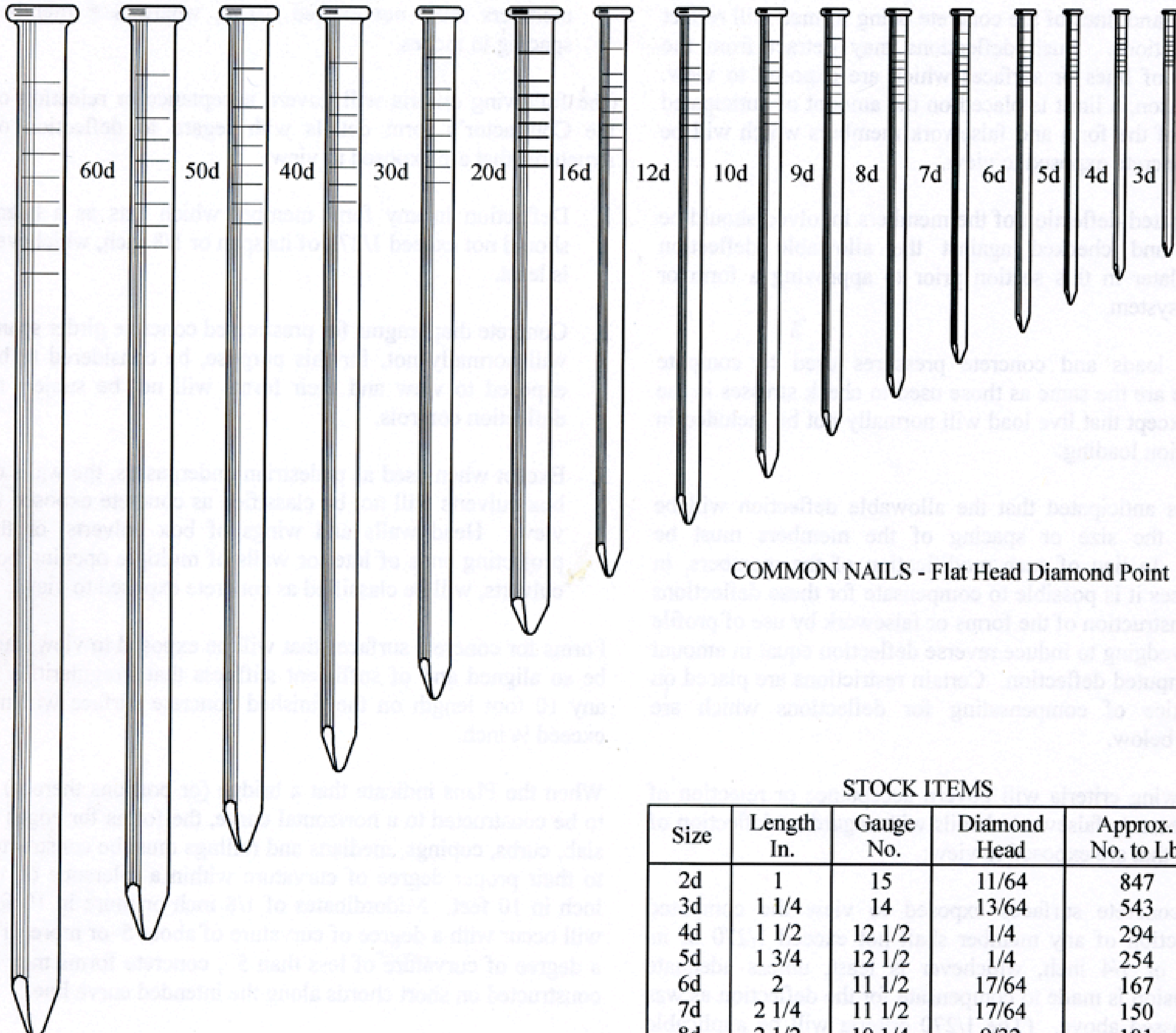


Figure 5-393-200-13– Details of some of the more commonly used form bolts and form ties.



STOCK ITEMS

Size	Length In.	Gauge No.	Diamond Head	Approx. No. to Lb
2d	1	15	11/64	847
3d	1 1/4	14	13/64	543
4d	1 1/2	12 1/2	1/4	294
5d	1 3/4	12 1/2	1/4	254
6d	2	11 1/2	17/64	167
7d	2 1/4	11 1/2	17/64	150
8d	2 1/2	10 1/4	9/32	101
9d	2 3/4	10 1/4	9/32	92
10d	3	9	5/16	66
12d	3 1/4	9	5/16	61
16d	3 1/2	8	11/32	47
20d	4	6	13/32	29
30d	4 1/2	5	7/16	22
40d	5	4	15/32	17
50d	5 1/2	3	1/2	13
60d	6	2	17/32	10

Length from underside of head to tip of point.

Figure 5-393-200-14– Diagram of the lengths and diameters of nails and spikes to be used to identify nails and spikes used in forms and falsework.

ALLOWABLE LATERAL LOADS ON NAILS

Driven into the side grain of seasoned wood. Load applied in any lateral direction.

SIZE PENNY WEIGHT	LENGTH (note 1) (in.)	DIAMETER "D" (in.)	*SAFE LATERAL LOAD ON EACH, IN LBS. At penetration in diameter noted for each group, into the piece holding the point.			
			GROUP I 10 X D	GROUP II 11 X D	GROUP III 13 X D	GROUP IV 14 X D
		COMMON NAILS (Flat Head, Diamond Point) Note (1) Length from underside of head to tip of point.				
6d	2	0.113	104	84	68	54
8d	2-1/2	0.131	129	104	86	68
10d	3	0.148	154	126	102	82
12d	3-1/4	0.148	154	126	102	82
16d	3-1/2	0.162	176	142	118	93
		SPIKS (Countersunk Head, Diamond Point) Note (1) Length overall				
10d	3	0.192	228	186	151	121
12d	3-1/4	0.192	228	186	151	121
16d	3-1/2	0.207	254	206	168	134
		DUPLEX HEAD NAILS (Heavy Double Head, Diamond Point) Note (1) Length from underside of lower head to tip of point.				
6d	1-3/4	0.113	104	84	68	54
8d	2-1/4	0.131	129	104	86	68
10d	2-3/4	0.148	154	126	102	82
16d	3	0.162	176	142	118	93
20d	3-1/2	0.192	228	186	151	121
30d	4	0.207	254	206	168	134
		SMOOTH BOX NAILS (Large Flat Head, Diamond Point) NOTE (1) Length from underside of head to tip of point.				
6d	2	0.099	84	68	56	44
7d	2-1/4	0.099	84	68	56	44
8d	2-1/2	0.113	104	84	68	54
10d	3	0.138	136	101	83	67

TIMBER SPECIES GROUPS
GROUP I Ash, Elm, Maple, Oak
GROUP II Douglas Fir, Larch, Southern Pine
GROUP III Hemlock Red Pine
GROUP IV Cedar, White & Balsam Fir, White Sugar Ponderosa and Lodgepole Pines Cottonwood, Spruce, Yellow Poplar

COOLERS (Flat Head, Diamond Point), SINKERS (Flat Counter Head, Diamond Point) as per BOX NAILS except length overall is 1/8" less than shown.

Basic Formulas: Safe Load = $1.33 \times K \times D^{3/2}$
where: Group I: K = 2040, Group II: K = 1650
Group III: K = 1350, Group IV: K = 1080

The values shown in this table, which are for normal load duration of 10 years have been increased by 1/3 due to short duration of static load on falsework.

Table 5-393-200-11– Allowable lateral loads on nails based on fastener diameter, length, and species of wood where used.

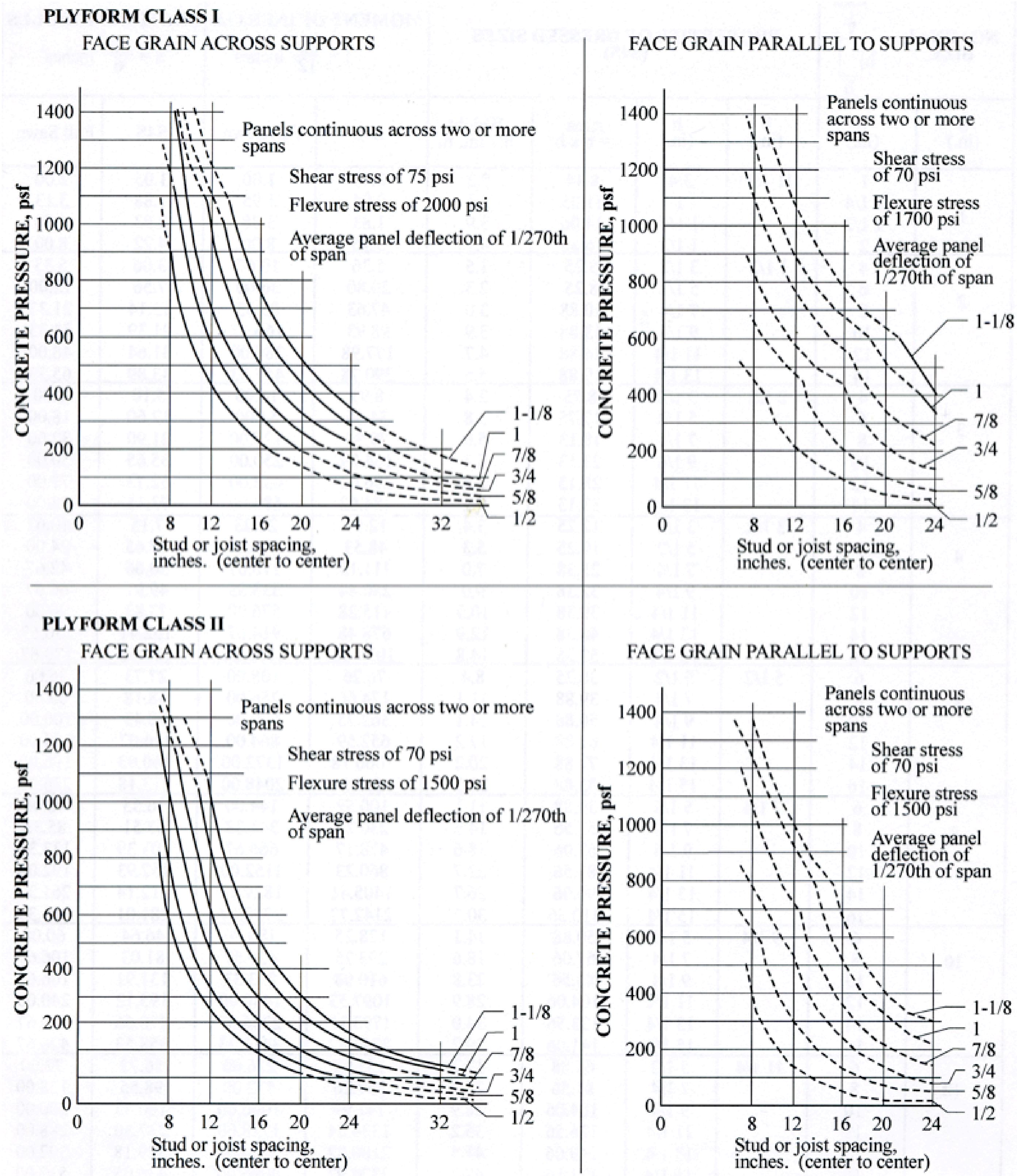


Figure 5-393-200-15– Diagrams used for determining the allowable uniform loads on Plyform based on Plyform thickness, spacing of supports, direction of face grain relative to direction of supports and grade of material.

Normally the manufacturer's literature will list the safe load that may be applied. However, when the load capacity is questionable or unknown, laboratory test are necessary to determine the safe load. In this event, the safe load may be calculated by determining the cross-sectional area of the member and the yield point of the steel by tension test in the laboratory. The applied load should not exceed 70% of the yield strength of the device. NOTE: The yield point of the steel (measured in psi) is not the same as the yield strength of a particular bolt or device.

On portions of the structure exposed to view, form bolts must be designed so that all the metal can be removed to a depth of not less than 1 inch. Wire ties may only be used in locations where they do not extend through surfaces exposed to view in the finished work.

The hardware used to secure form bolts against the forms is usually reusable. This hardware is normally designed to be stronger than the portion of the device that remains in the concrete and therefore will not be the limiting factor in the form tie.

Crimp ties or snap ties are wire form ties with a notch or reduced cross-section at the point of break-back. These ties are not reusable. After the concrete is set, the portion of the wire, that extends outside of the concrete surface is twisted off and removed. A washer is sometimes welded to the wire at the face of the form to act as a form spreader. On concrete surfaces exposed to view, a plastic cone should be used in place of the washer since satisfactory patching of the shallow depression left by the washer is very difficult.

These ties do not always break off at the intended point, but sometime break instead at the face of the concrete. They also do not provide a rigid member for the support of workers, for these reasons they are not recommended for use on heavy construction. Their use is primarily restricted to light work such as box culverts.

5-393.205 Engineering Mechanics

Engineering Mechanics is the science that considers the action of forces on material bodies. Statics considers forces in equilibrium or of bodies held in equilibrium by the forces acting on them. Kinetics considers the relations between forces on bodies in motion. This Chapter only deals with bodies held in equilibrium, bodies in which there is no motion. Forces on forms and falsework are the results of loads applied to the structural elements of the formwork.

When a force acts on a body, two things happen. First, internal forces that resist the external forces are set up in the body. These internal resisting forces are called stresses. Secondly, the external forces produce deformation or changes in shape of the body.

Strength of material is the study of the properties of material bodies that enable them to resist the action of external forces, the study of the stresses within the bodies and deformations that result from the external forces.

Loads applied to structural members may be of various types sources. Most of the loads on forms and falsework are considered to be static loads. These are loads that are gradually applied and equilibrium is reached in a relatively short time. An example of these types of loads are the loads and pressures from the fresh concrete being restrained by the form members and the live load from the workers placing and finishing the concrete. These loads are considered static loads. Loads that are constant over a long period of time are considered sustained loads. The earth pressure behind the main wall of bridge abutments is an example of a sustained load. Forms and falsework are rarely subjected to sustained loads. A load that is rapidly applied is called am impact load. Forms and falsework are normally very rarely subjected to impact loads. Forms and falsework are designed to resist loads in the middle ground between impact loads and long-term sustained loads.

Loads are also classified with respect to the area over which the load is applied. A concentrated load is one that can be considered to be applied only at a point. Any load that is applied to a relatively small area can be considered a concentrated load. This could be the load from a single joist resting on a beam. A distributed load is a load that is distributed along the length of a beam or spread over an area. The distribution may be uniform or non-uniform. Non-uniform loads are sometime referred to as uniformly varying loads. An example of an non-uniformly varying load is the weight of the concrete in the overhang of most bridges, where it is thinner at the outside edge of the bridge, the coping, and thicker at the fascia beam.

Loads can be further classified with respect to the location and method of application to structural members. A load in which the resultant concentrated load passes through the centroid (geometrical center) of the resisting cross-section is referred to as centric loads. If the resultant concentrated forces passes through the centroids of all of resisting sections, the loading is called axial. A load that is applied transversely to the longitudinal axis of the member is referred to as a bending or flexural load. A member subjected to bending loads deflects along its length. A load that subjects a member to a twisting moment is called a torsional load. The load that a bridge deck overhang bracket applies to the fascia beam of a bridge is an example of a torsional load. A condition where two or more of the types of loads as previously described act simultaneously is called a combined load.

A. Concept of Stress

Stress, like pressure, is a term used to describe the intensity of a force. Stress is the quantity of a force that acts on a unit of area. Direct stresses are those stresses that are uniform over the entire cross section of the member. Force, in structural design, has little significance until something is known about the resisting material, cross-sectional properties and the size of the element resisting the force. The unit stress, the average value of the axial stress, may be represented mathematically as:

$$f = \frac{P}{A} = \frac{axial\_force}{perpendicular\_resisting\_area}$$

where:

f = the symbol(s) representing unit stress; usually expressed as pounds per square inches or psi

P =applied force or load (axial); usually expressed in pounds, lb

A = resisting cross-sectional area perpendicular to load direction, units normally are square inches or in^2

B. Standard Notation

All of the examples and explanations for those examples contained in this Chapter are based on the standard notation used in the Allowable Stress Design (ASD) procedure. The use of standard notation is important for a number of very good reasons. There are two reasons that deserve amplification. First, there are two generally accepted design procedures, Allowable Stress Design (ASD) and Load and Resistance Factored Design (LRFD), the former gradually being replaced by the later. Each of these two design methods has developed their own distinctive format to their calculations. The second advantage of adhering to the conventions in each of the standard notations is that it is much easier for reviewers to understand calculations and avoids misunderstandings.

As stated above, all of the examples and explanations utilize the standard notations of ASD procedures. As the name implies this method of design uses stress created by the actual loads and compares them against the Allowable Working Stress. The actual stresses calculated in all of the different members are abbreviated using the italic lower case letter *f*. This lower case letter *f*, is almost always followed by a subscript letter, such as *b*, *v*, *c* that indicates what type of stress in involved. For example a calculated bending stress will be noted as f_b = 1,300 psi. This specific notation tells us that the calculated bending stress in this particular member is 1,300 pounds per square inch. If the stress is denoted as f_v, this is a shear stress.

The calculated bending moment in a member is always denoted with an upper case italic letter *M*. The section modulus of a structural member is always denoted with an upper case letter italic *S*. The cross-sectional area of a member is denoted with an upper case italic letter *A*.

Stresses identified as material properties are always denoted with a upper case italic letter *F*, additionally the upper case letter *F* has a subscript letter that indicates the category of stress, such as $_b$ for bending stress, $_v$ for shear stress, and $_c$ for compressive stress. Also, in addition to the subscript letter the upper case *F* is sometimes followed by an apostrophe that indicates that the stress is the fully adjusted working stress. For example, the allowable bending stress would be denoted as F_b' and the allowable shear stress is denoted by F_v'.

C. Types of Stress

There are few basic ways that material resists forces imposed on them. Those forces are then resisted by the stresses resulting in the material. Those cases where the stresses are uniform over the cross-sectional area are referred to as direct stresses. The four common direct stresses are as follows:

1. Compressive stress
2. Tensile stress
3. Bearing stress
4. Shear stress

Compressive Stress:

Compressive stress results from forces on a structural element that tends to shorten that element. Structural elements that typically resist compressive forces are columns and piles. In timber members compressive stress in normally assumed to be acting parallel to the direction of grain of the wood in the member. Compressive stress can be represented by the following formula:

$$f_c = \frac{P}{A}$$

where:

f_c = unit compressive stress, psi

P = Applied load, lb

A = Resisting area normal (perpendicular) to P, in^2

Tensile Stress:

Tensile stress results from forces on a structural element that tend to stretch or lengthen that element. The bottom chord of a typical truss is a common tensile element. Tensile stress can be represented by the following formula:

$$f_t = \frac{P}{A}$$

where:

f_t = unit tensile stress, psi

P = applied tensile load, lbs

A = resisting area normal (perpendicular) to P, in^2

Bearing Stress:

Bearing stress is the resulting stress from forces that are acting perpendicular to the exposed surface of the member. These stresses result from bearing plates or under washers. In timber structural elements this type of stress is assumed to be acting perpendicular to the direction of grain of the wood within the member. This type of compressive stress can be represented by the following formula:

$$f_p = \frac{P}{A}$$

where:

f_p = unit bearing stress, psi

P = applied load, lb

A = bearing contact area, in^2

Shear Stress:

A shearing stress results when two parallel forces having opposite directions act on a body, tending to cause one part of the body to slide past an adjacent part. One common example is that of a horizontal beam supporting a load. The magnitude of the shear is represented by *V*; because in most cases the shear forces are acting vertically. The vertical shear in a horizontal beam varies in magnitude at different locations along the beam's length and is easily visualized when shown in a shear diagram. The actual shear stress in steel and concrete structural elements can be represented by the following formula:

$$f_v = \frac{V}{A}$$

where:

f_v = unit shear stress, psi

V = vertical shear, lb

A = cross-sectional area parallel to load direction, lbs

The calculation of shear stress in wood members requires different considerations. Wood is much weaker in shear resistance along the direction of grain. Therefore, when wood fails in shear it will shear along the direction of grain. The determination of the vertical shear, *V*, is the same for wood members as for steel and concrete members. It is assumed that the shear force, *V*, is acting perpendicular to the direction of the grain in the wood member. The actual shear stress in timber bending members can be determined with the following formula:

$$f_v = \frac{3V}{2bd}$$

where:

f_v = unit shear stress in wood member, psi

V = vertical shear force, lb

b = width of wood member, in

d = depth of wood member, in

D. Material Properties

The general term *material properties* refers to several aspects of a material to resist forces. The *strength* of a material is its ability to resist the three basic stresses; compression, tension, and shear. Some materials exhibit different strength properties for each of the different types of stress to which the material is subjected. As an example, the compressive and tensile strength of structural steel are about equal, whereas cast iron is strong in compression and relatively weak in tension. Additionally, wood exhibits much different strength characteristics not only for the different types of stress, but different strength values relative to the direction of the force with respect to the direction of the grain of the wood.

The *ultimate strength* of a material is the unit stress that causes failure or rupture. The term *elastic strength* is generally applied to the greatest unit stress a material can resist without a permanent change in shape.

The *stiffness* of a material is that property that enables it to resist deformation. If, for instance, blocks of steel and wood of equal size are subjected to equal compressive loads, the wood block will become shorter than the steel block. The deformation (shortening) of the wood block will probably be about 20 time that of the steel, and we say, the steel is *stiffer* than the wood.

Elasticity is that property of a material that enables it to return to its original size and shape when the load to which it has been subjected is removed. This property varies in different materials. This property is called the *modulus of elasticity* and is represented by *MOE* or *E*. For certain materials there is a unit stress beyond which the material does not regain its original dimensions when the load is removed. The magnitude of this unit stress is called the *elastic limit* of that material. In many cases, the allowable working load unit stresses for such a material should be well below the elastic limit. Every material changes its size and shape when subjected to loads. For the materials used in bridge construction the actual unit stresses should be such that the deformations for direct stresses are in direct proportion to the applied loads. Or in other words, the working stresses should be below the elastic limit of the material.

Plasticity is the opposite quality to elasticity. A perfectly plastic material is a material that does not return to its original dimensions when the load causing the deformation is removed. There are probably no perfectly plastic materials. Modeling clay and lead are examples of plastic materials.

Ductility is that property of a material that permits it to undergo plastic deformation when subjected to a tensile force. A material that may be drawn into wires is a ductile material. A chain made of ductile material is preferable to a chain in which the material is brittle.

A material having the property that permits plastic deformation when subjected to a compressive force is a *malleable* material. Materials that may be hammered into sheets are examples of malleable material. Ductile materials are generally malleable. A material, such as cast iron, for instance, that is neither malleable nor ductile is called *brittle*.

Whenever a body is subjected to a force there is always a change in the shape or size of the body. These changes in dimensions are called *deformations*. A block subjected to a compressive force *shortens* in length and the decrease in length is its deformation. When a tensile force is applied to a rod, the original length of the rod is increased and the *lengthening* or *elongation* is its deformation. A loaded beam resting on two supports at its ends tends to become concave on its upper surface; we say the beam *bends*.

The deformation that accompanies the bending is called *deflection*. The amount of deflection can be expressed in two ways. First, the actual deflection can be measured in units of length, generally in inches. The amount of deflection an also be expressed as a ratio of the span length. This is sometimes referred to as the *span-to-deflection (L/D)* ratio. It is very common for the actual amount of deflection for a loaded beam be limited to some ratio, such as 1/240 of the span length. Sometimes the deflection of a beam is represented by the Greek letter delta, Δ, the span-to-deflection ratio might be shown as *L/*Δ.

Strain:

When stresses occur in a body, there is always an accompanying deformation. The deformation often is so small that it is not apparent to the naked eye; nevertheless, it is always present. The term *strain* is sometime used as a synonym for deformation. The relationship between stress and strain is a key concept. The terms stress and strain can and will be used in a couple of different forms, it can be shown as *stress/strain*, or as *stress and deformation*, all of which refer to the same concept.

The modulus of elasticity, as stated earlier, is a measure of the stiffness of a material. The modulus of elasticity is defined as the ratio of the unit stress to the unit deformation. It is generally represented by the letter *E* and is a measure of the stiffness of a material. It can berepresented mathematically by the following formula:

$$E = \frac{unit\_stress}{unit\_deformation}$$

Thus the modulus of elasticity is the unit stress divided by the unit deformation. Further, since the unit stress is in units of pounds per square inch (psi) and the unit deformation is an abstract number (inches divided by inches); therefore, the modulus of elasticity is in units of pounds per square inch.

From previous definitions we can express modulus of elasticity in several different ways as follows:

$$E = \frac{unit\_stress}{unit\_deformation} = \frac{f}{s}$$

$$\frac{f}{s} = \frac{P/A}{e/l} = \frac{P}{A} \div \frac{e}{l} = \frac{P}{A} \times \frac{l}{e} = \frac{Pl}{Ae} \therefore$$

$$e = \frac{PL}{AE}$$

where:

E = modulus of elasticity, psi

P = applied force, lb

f = unit stress in member, psi

A = area of cross section, in^2

l = length of member, in

e = total deformation, in

s = unit deformation in/in

By these formulas, we are able to determine the deformation of a member subjected to stresses, provided that we know the modulus of elasticity of the material and the unit stress does not exceed the elastic limit of the material.

Cross Sectional Properties:

The design and understanding of members that are subjected to loads that tend to bend the member require knowledge in several additional areas. Unlike members subjected to compressive, tensile, and shear stresses members subjected to bending loads do not have uniform stresses spread over the cross sectional area. The stresses from bending loads are concentrated at various locations and the magnitude of those stresses are not only related to the size of the member, but are also related to the shape of the cross sectional area.

The shape and proportion of a beam's cross section is critical in keeping the bending and shear stress within the allowable limits and moderating the amount of deflection that will result from the loads. Why does a 2" x 8" joist standing on edge deflect less than when loaded at mid-span than the same 2" x 8" when used flat-wise as a plank? The difference in performance of the same structural element is controlled by the different section properties of the piece in two different positions.

There are four different section properties for every structural member that are used for structural analysis. These four section properties are:

1. area
2. center of gravity (centroid)
3. section modulus
4. moment of inertia

The area of the cross section of most structural members used in formwork can easily be calculated or found in any one of many readily available tables. The *center of gravity* of a solid is an imaginary point at which all of its weight may be considered to be concentrated or the point through which the resultant weight passes. Since an area has no weight, it has no center of gravity. The point of a plane area that corresponds to the center of gravity of a very tin homogeneous plate of the same area and shape is called the *centroid* of the area.

Bending and Shear Stress in Beams:

When a simple beam is subjected to forces that tend to cause it to bend, the fibers above a certain plane in the beam are in compression and those below the plane are in tension. This plane is called the *neutral surface*. For a cross section of the beam the line corresponding to the neutral surface is called the *neutral axis*. The neutral axis passes through the centroid of the cross section; thus it is important that we know the exact position of the centroid of any structural member we are using. The position of the centroid for a symmetrical shape is readily determined.

The moment of inertia of an area is an abstract term that is the summation of the products of all of the tiny areas multiplied by the square of the distance from the neutral axis. It is normally represented by the letter I and is defined by the following equation:

$$I = \sum az^2$$

where:

I = moment of inertia, in^4

a = infinitely small areas, in^2

z = distance from neutral axis to centroid of small areas, in

There are numerous tables available that give the section properties, including moment of inertia, for almost all common construction materials. Additionally, there are many sources that provide the formulas for calculating the moment of inertia for most common shapes of cross sectional areas. For instance, the formula for calculating the moment of inertia for members with rectangular cross section is:

$$I_{x-x} = \frac{bd^2}{12}$$

where:

I_{x-x} = moment of inertia relative to x-x axis, in^4

b = width of beam, in

d = depth of beam, in

One of the properties of cross sections constantly used by designers is a quantity called the *section modulus*. Its use in the design of beams will be explained later, for the present it is only necessary to know that if I is the moment of inertia of a cross section about an axis passing through the centroid and if c is the distance of the most remote fiber of the cross section from the neutral axis, the section modulus equals:

$$S = \frac{I}{c}$$

where;

S = section modulus, in^3

I = moment of inertia, in^4

c = distance from neutral axis to most remote fiber, in

Using the formula for the moment of inertia for a rectangular beam and the relationship of the section modulus for the same cross section we can derive the formula for the section modulus as follows:

$$S = \frac{bd^3}{12} \div \frac{d}{2} = \frac{bd^3}{12} \times \frac{2}{d} \therefore$$

$$S = \frac{bd^2}{6}$$

where:

S = section modulus, in^3

b = width of beam, in

d = depth of beam, in

Again, there are many tables available that list all of the section properties for most common construction materials. **Table 200-2** lists the section modulus and moment of inertia for most common sizes of lumber.

There are a number of tools and techniques that have been developed over the years that can be used in the analysis of beams. One of the most useful techniques for the examination of structural members is the use of a *free body diagram* (FBD). A free body diagram is nothing more than a simplistic diagram of the member showing only the essential information in its relative position. These diagrams are very useful in determining which of all of the available design aids are appropriate to use in the solution of the problem.

There are many FBD published that cover a great many of the possible configurations for beams. These published diagrams are called "Beam Diagrams and Formulas", the American institute of Steel Construction (AISC) in their *Manual of Steel Construction* publishes the most complete listing. That publication also contains a complete listing of the section properties of rolled steel shapes. The published diagrams and formulas provide formulas for the calculation of the maximum bending moments, the maximum shear, and the deflections associated with the given loading conditions. See **Figure 200-16** for beam formulas for some commonly used load cases. There are other design aids that can be used in the analysis and checking of forms and falsework plans. Three design aids for calculating the effective bearing areas for different configurations are shown in **Figure 200-17.**

5-393.206 Concrete Pressure

Concrete exerts loads on forms and falsework in two general ways. First, concrete exerts dead load anywhere the fresh concrete is restrained vertically. The magnitude of this load does not change over time. The second way in which fresh concrete exerts loads on formwork is from the lateral pressure from the concrete in a plastic state. This occurs any place where the fresh concrete is restrained horizontally.

When concrete is first mixed, it has properties lying between a liquid and a solid. Fresh concrete is generally defined as a plastic material. As time passes from the initial time of mixing, concrete loses its plasticity and gradually changes into a solid. This change from a semiliquid state to a solid state is the result of two actions within the concrete.

The first action results from the setting of the cement, which may start within 30 minutes after the concrete is mixed. This action may continue for several hours dependent on the temperature. The warmer the temperature the quicker the setting occurs. The other action is the development of internal friction between the particles of aggregate in the concrete that restrains them from moving freely with respect to other particles in the mixture.

A. Concrete Lateral Loads

The American Concrete Institute (ACI) has developed methods for the calculation of lateral pressure from concrete. The pressure exerted laterally on forms is controlled by several or all of the following factors:

1. Rate of placing concrete in forms
2. Temperature of concrete
3. Weight or density of concrete
4. Cement type or blend used in the concrete
5. Method of consolidating concrete
6. Method of placement of the concrete
7. Depth of placement
8. Height of form

ACI has identified the maximum pressure on formwork as the full hydrostatic lateral pressure, as given by the following equation:

$$P_m = wh$$

where:

P_m = maximum lateral pressure, psf

w = unit weight of placed concrete, pcf

h = depth of plastic concrete, ft

For concrete that is placed rapidly, such as columns, h should be taken as the full height of the form. There are no minimum values given for the pressure calculated above. The maximum hydrostatic lateral pressure as defined above is generally not used for the design of concrete forms (see next section).

For the purpose of structural design of forms for vertical structural elements are separated into one of two categories based on the configuration of the elements, namely; walls and columns. A wall section is defined as a vertical structural element with at least one plan-view dimension greater than 6.5 feet. Those vertical structural vertical elements with all dimensions in a plan-view less than 6.5 feet. are classified as columns.

B. Lateral Pressure from Concrete on Wall Forms

There are two formulas for calculating the lateral pressure from concrete on wall forms. The first equation applies to a wall with a rate of placement less than 7 feet per hour and a placement height of 14 feet or less. The other equation applies to all walls with a placement rate of 7 to 15 feet per hour, and to walls placed at less than 7 feet. per hour but having a placement height greater than 14 feet. Both equations apply to concrete with a 7 inch maximum slump. For walls with a rate of placement greater than 15 feet per hour, or when forms will be filled rapidly, before any stiffening of the concrete occurs, then the pressure should be taken as the full hydrostatic pressure given by the following formula, $P_m = wh$ (see section above).

For wall forms with a concrete placement of less than 7feet per hour and a placement height not exceeding 14 feet lateral pressure can be calculated by the following formula with a minimum of 600C_W psf, but in on case greater than wh.

$$P_m = C_W C_C \left[150 + 9{,}000 R / T\right]$$

Where:

P_m = maximum lateral pressure, psf

C_w = unit weight coefficient as given in **Table 5-393-200-13**

C_c = chemistry coefficient as given in **Table 5-393-200-12**

CHEMISTRY COEFFICIENT, C_C

Admixtures - Retarders, Water Reducers	C_C
Types 1, II and III without retarders	1.0
Types 1, II and III with a retarder	1.2
Other types or blendswithout retarders* containing less than 70% slag or less than 40% fly ash	1.2
Other types or blend with a retarder* containing less than 70% slag or less than 40% fly ash	1.4
Blends containing more than 70% slag or 40% fly ash	1.4

* Retarders include any admixtures such as a retarder, retarding water reducer, retarding mid-range water reducing admixture, or retarding high-range water reducing admixtures (superplasticizer)that delays setting of concrete.

Table 5-393-200-12– Chemistry Coefficient, C_C to be used with the formulas for calculation of lateral pressure from fresh concrete.

UNIT WEIGHT COEFFICIENT, C_W

Concrete Unit Weights
Concrete weighing less than 140 pcf $C_W = 0.5(1 + w/145)$ but not less than 0.80
Concrete weighing 140 to 150 pcf $C_W = 1.0$
Concrete weighing more than 150 pcf $C_W = w/145$

Table 5-393-200-13– Unit Weight Coefficient, C_W to be used with the formulas for calculation of lateral pressure from fresh concrete.

R = rate of fill of concrete, ft. per hour

T = temperature of concrete, degrees F°

Minimum value of P_m is $600C_w$, but in no case greater than *wh*.

For all wall forms with concrete placement rate from 7 feet to 15 feet per hour, and for walls where the placement rate is less than 7 feet per hour and the placement height exceeds 14 feet lateral pressure can be calculated by the following formula:

$$P_m = C_W C_C \left[150 + \frac{43{,}400}{T} + 2{,}800 R / T\right]$$

where:

P_m =maximum lateral pressure, psf

C_w = unit weight coefficient as given in **Table 5-393-200-13**

C_c = chemistry coefficient as given in **Table 5-393-200-12**

R = rate of placement of concrete, ft. per hour

T = temperature of concrete, degrees F°

Minimum value of P_m is $600C_w$, but in no case greater than *wh*.

C. Lateral Pressure from Concrete on Column Forms

The ACI methods for determining the pressure on formwork makes a distinction in vertical structural elements based on the greatest plan-view dimension. Those vertical elements that do not have a plan-view dimension greater than 6.5 ft. are classified as a column. Additionally, the ACI recommends that formwork be designed for the full hydrostatic pressure in situations where the concrete is placed rapidly and where self-consolidating concrete is used. In the case of tapered pier shafts, the minimum plan-view dimension will govern in determining the classification of the vertical element as either a wall section or column.

Calculation of the lateral pressure for concrete with a slump of 7 inches or less and placed by normal internal vibration can be based on the following formula:

$$P_m = C_W C_C \left[150 + \frac{9{,}000 R}{T}\right]$$

where:

P_m = calculated lateral pressure, psf

C_w = unit weight coefficient as given in **Table 5-393-200-13**

C_c = chemistry coefficient as given in **Table 5-393-200-12**

R = rate of placement of concrete, ft. per hour

T = temperature of concrete, F°

Minimum value of P_m is $600C_w$, but in no case greater than *wh*

With modern techniques of placing and intensive vibration, it is possible with rapid rates of filling forms to have concrete remaining in a fluid condition for the full duration of the pour, in which case theoretically the only pressure limit will be *wh*.

5-393.207 Engineering Analysis

Practically all falsework members act either as columns or as beams. If the members are called elements, both words can be used interchangeably. The internal stresses and deflections in these members are due to the effects of the various construction loads. These loads are divided into two distinct categories, namely, **dead loads** and **live loads.** Dead loads consist of the weight of the actual forms and falsework, the weight of the concrete, and the pressure produced by the fresh concrete. Live loads are from the weight of the work crews and the equipment used in the construction and used for placements and finishing of the concrete. The combined effect of all of these various loads on structural members in the forms and falsework can be determined by standard engineering mechanics as covered earlier in section

5-393.206 Engineering Mechanics. The stresses produced by these loads are then compared to the allowable working stresses for each member to determine the adequacy of the structural design of the forms and falsework.

A. Important Background Information:

The contract provisions of some bridge projects will require the Contractor to provide plans for his proposed forms and falsework that have been prepared and certified by a licensed engineer. Contract provisions for other less complicated bridges may not require sealed falsework plans. Whether or not the contract requires the Contractor to provide plans sealed by an Engineer, does not in any way relieve the Contractor of total responsibility for the performance of the forms and falsework.

The roles and responsibilities of the engineer can change in the review of the design and inspection of the completed formwork. If the Contract provisions require the Contractor to provide plans for the forms and falsework the Engineer will have the benefit of a plan to guide his inspection. However, whether or not a sealed plan is required to be furnished on any particular bridge, it is extremely important that the Engineer have some understanding of the analysis of forms and falsework.

The information presented in this Chapter has been drafted in two important ways to help the Engineer develop a basic understanding of the subject. First, all of the information presented in both the text portion of the manual and in the example problems uses "Standard Notation" through out this Chapter. This is done to emphasize the similarities between many of the various terms, principals, properties and values. Next, wherever possible the procedures have been simplified. Simplifications were generally done in the conservative direction, which results in solutions that are less efficient than results arrived at using more rigorous analytic methods. The purpose of this Chapter is to provide Engineers with a basic understanding of the process and the ability to check some aspects of forms and falsework plans, not to become falsework Engineers.

The Engineer must be satisfied that the Contractor's falsework plan or scheme is in conformance with the *Standard Specifications for Construction* as published by the Minnesota Department of Transportation. A common method to do this is to compute the maximum deflection and maximum stresses (bending, bearing, compression, shear, etc.) based on plans of the proposed falsework and assumed loading conditions. These stresses and deflections are then compared with the allowable values. If the computed stress and deflection is less than or equal to the allowable value, the member qualifies for use.

B. Deflection and Alignment:

Deflection will occur in **all** form and falsework members in which beam action is involved, regardless of the design used or the material of which the forms and falsework are constructed. The surfaces and lines of the concrete being formed will reflect these deflections. Such deflections may detract from the appearance of lines and surfaces that are exposed to view. For this reason, a limit is placed on the amount of anticipated deflection of the forms and falsework members, which will be applied for concrete surfaces exposed to view.

The anticipated deflection of members involved should be computed and checked against the allowable deflection as described later in this section prior to approving a form or falsework system. The loads and concrete pressures used to compute deflection are the same as those used to calculate the stresses in the members, except that the live load will normally not be included in the loading used for deflection.

When it is anticipated that the allowable deflection will be exceeded, the size or spacing of the members must be modified. In lieu of such modification of the members, in certain cases it is possible to compensate for these deflections during construction of the forms and falsework by the use of profile strips or wedging to induce reverse deflection equal to the amount of the computed deflection. This reverse deflection used to off-set anticipated deflection is called "camber." Certain restrictions are placed on this practice of compensating for deflection, which will be discussed below.

1. Falsework Deflections for Surfaces Exposed to View: The following criteria will govern acceptance or rejection of the Contractor's falsework details with regard to deflection:

(a.) On concrete surfaces exposed to view, the computed deflection of any member shall not exceed 1/270 of its span or ¼ inch, whichever is least., unless adequate provision is made to compensate for the deflection as was described above. (The 1/270 criteria will be applied for spans up to 67 inches.)

(b .) Between fascia beams, the falsework supporting the deck slab will not be limited by the foregoing. In this area, a limiting cumulative deflection (deflection of sheathing plus deflection of stringers plus deflection of the joists, etc.) of ½ inch should be applied. This

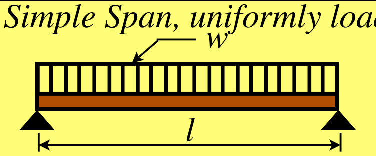
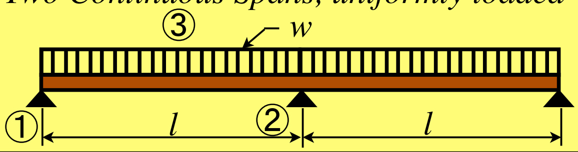
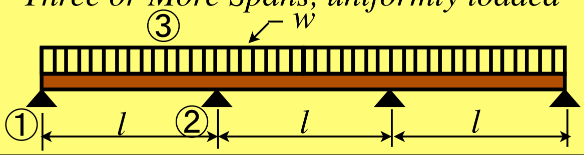
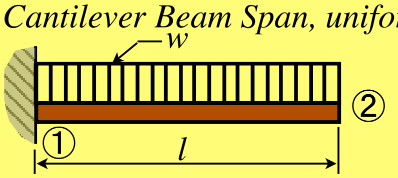
Load Case	max. R	max. M	max. V	max. Δ
Simple Span, uniformly loaded 	$\frac{wl}{2}$	$\frac{wl^2}{8}$	$\frac{wl}{2}$	$\frac{5wl^4}{384EI}$
Two Continuous Spans, uniformly loaded 	$R_1 = \frac{3wl}{8}$ $R_2 = \frac{5wl}{4}$	$M_3 = \frac{wl^2}{14.2}$ $M_2 = \frac{wl^2}{8}$	$V_1 = \frac{3wl}{8}$ $V_2 = \frac{5wl}{8}$	$\Delta_3 = \frac{wl^4}{185EI}$
Three or More Spans, uniformly loaded 	$R_1 = 0.4wl$ $R_2 = 1.1wl$	$M_3 = 0.08wl^2$ $M_2 = 0.10wl^2$	$V_1 = 0.4wl$ $V_2 = 0.6wl$	$\Delta_3 = \frac{0.0069wl^4}{EI}$
Cantilever Beam Span, uniformly loaded 	$R_1 = wl$	$M_1 = \frac{wl^2}{2}$	$V_1 = wl$	$\Delta_2 = \frac{wl^4}{8EI}$

Figure 5-393-200-16– Beam formulas for calculating reactions, moments, shears and deflections based on different configurations and loading conditions.

limit is to avoid excessive additional dead load weight to the superstructure.

(c.) At locations of transverse construction joints in the roadway slab, the falsework supporting the bulkhead must be sufficiently strong to reduce the computed bulkhead deflection to no more than 1/16 inch.

(d.) The deflection of slab overhang falsework must be compensated for by wedging or raising the outside edge of the overhang falsework by an amount equal to that of the anticipated deflection. The anticipated cumulative deflection of the overhang falsework must not exceed ½ inch even though compensated for. When the main overhang falsework supporting members (the overhang bracket, needle beams or equivalent systems) are spaced at least 48 inches, the anticipated deflection of these members must not exceed $S/100$, where S = member spacing in inches.

2. **Falsework Deflections for Surfaces Not Exposed to View:** The following criteria will govern acceptance of the Contractor's form details with regard to deflection:

(a.) Deflection in any form member that acts as a beam should not exceed 1/270 of its span or 1/8 inch, whichever is least.

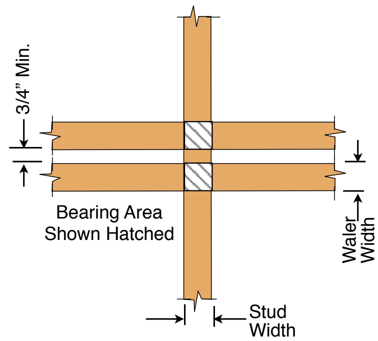
(b.) Concrete diaphrams for prestressed concrete girder spans will normally not, for this purpose, be considered to be exposed to view, and their forms will not be subjected to deflection controls.

(c.) Except when used as pedestrian underpasses, the walls of box culverts will not be classified as concrete exposed to view. Head walls and wing walls of box culverts, or the projecting ends of interior walls of multiple opening box culverts, will be classified as concrete exposed to view.

3. **Form Alignment:** The following criteria will govern for the acceptance of the Contractor's form details with regard to alignment:

CONTACT AREAS AND ALLOWABLE STRESS INCREASE FACTORS

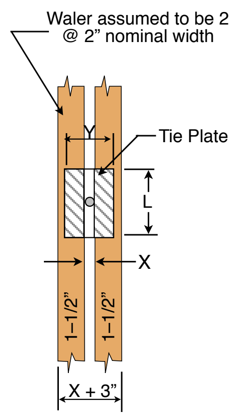
STUD WIDTH (in.)		WIDTH OF ONE WALLER (in.) 1-1/2	2	2-1/2	3	3-1/2	4
1-1/2	(1)	4.50	6.00	7.50	9.00	10.50	12.00
	(2)	1.25	1.19	1.15	1.13	1.11	1.09
2	(1)	6.50	8.00	10.00	12.00	14.00	16.00
	(2)	1.19	1.19	1.15	1.13	1.11	1.09
2-1/2	(1)	7.50	10.00	12.50	15.00	17.50	20.00
	(2)	1.15	1.15	1.15	1.13	1.11	1.09
3	(1)	9.00	12.00	15.00	18.00	21.00	24.00
	(2)	1.13	1.13	1.13	1.13	1.11	1.09
3-1/2	(1)	10.50	14.00	17.50	21.00	24.50	28.00
	(2)	1.11	1.11	1.11	1.11	1.11	1.09
4	(1)	12.00	16.00	20.00	24.00	28.00	32.00
	(2)	1.09	1.09	1.09	1.09	1.09	1.09



CONTACT AREAS AND ALLOWABLE STRESS INCREASE FACTOR FOR WALERS AND TIE PLATES

L (in.)	Y (in.)	X (in.)	CONTACT AREA (sq. in.)	ALLOWABLE STRESS INCREASE FACTOR, C_b
3-1/4	3-3/4	3/4	9.75	1.12
5	3-1/4	3/4	12.50	1.08
3-3/4	3-1/2	3/4	10.31	1.10
3	*3-3/4	3/4	9.00	1.13
5	*3/3/4	3/4	15.00	1.08
5-1/4	*3-3/4	3/4	15.75	1.07
5-3/4	*3-3/4	3/4	17.25	1.00
6	*3-3/4	3/4	18.00	1.00
6-1/4	*3-3/4	3/4	18.75	1.00
6-3/4	*3-3/4	3/4	20.25	1.00
5	3-3/4	1	13.75	1.08
5	4	1	15.00	1.08
5-1/4	4	1	15.75	1.07
5-3/4	4	1	17.25	1.00
6	4	1	18.00	1.00
6-1/4	4	1	18.75	1.00
6-3/4	4	1	20.25	1.00

*or more



BEARING AREA IN SQ. IN. BETWEEN CAPS AND PILES OF VARIOUS SIZES

Diameter of Pile at Cut-off (in.)	(Piles assumed to be circular) ACTUAL WIDTH OF CAP IN INCHES 6	7-1/2	8	9-1/2	10	11-1/2	12	13-1/2	14
14	81.4	99.7	105.6	121.9	127	140.4	144.2	152.7	153.9
13-1/2	78.2	95.8	101.3	116.7	121.4	133.6	136.9	143.1	
13	75.1	91.8	97	111.4	115.7	126.6	129.4	132.7	
12-1/2	72	87.8	92.7	106	109.9	119..4	121.5	122.7	
12	68.9	83.7	88.3	100.6	104.1	111.9	113.1		
11-1/2	65.7	79.7	83.9	95.1	98.1	103.9			
11	62.6	75.6	79.5	89.4	91.9	95			
10-1/2	59.4	71.4	75	83.6	85.5	86.6			
10	56.2	67.2	70.4	77.5	78.5				
9-1/2	52.9	62.9	65.7	70.9					
9	49.7	58.6	60.8	63.6					
8-1/2	46.4	54	55.8	56.7					
8	43	49.3	50.3						

NOTE: Bearing area at right end of each line is the area of pile at cut-off of the diameter shown at left. Use when cap width equals or exceeds pile diameter.

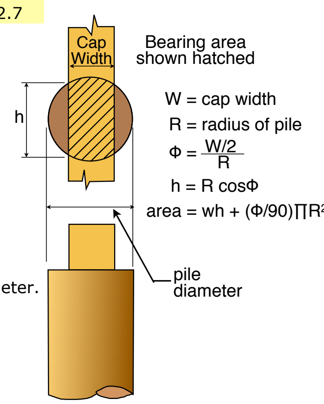


Figure 5-393-200-17– Engineering and design information for determining the contact area and bearing stress adjustment factors for several different common construction connections.

(a.) Deflection in any form member that acts as a beam should not exceed 1/270 of its span or 1/8 inch, whichever is least.

(b.) When the Plans indicates that a bridge (or portions thereof) is to be constructed in a horizontal curve, the forms for the edges of the slabs, curbs, copings, medians and railings must be constructed to their proper degree of curvature within a tolerance of 1/8 inch in 10 feet. Mid-ordinates of 1/8 inch or more in 10 feet will occur with a degree of curvature of about 5 Degrees or more. For a degree of curvature of less than 5 Degrees, concrete forms may be constructed on short chords along the intended curve line. It is intended that forms that can easily be placed to a scribed line on the falsework or on previously placed concrete will be placed on the specified curved alignment. This would include forms for the edge of slabs, curbs, and medians.

(c.) No offsets should exist at abutting joints of sheathing or at abutting form panels.

(d.) The variation from plumb or from the specified batter in the line and surfaces of columns, piers, and walls, should not exceed ¼ inch per 10 feet of height and in any event shall not exceed ½ inch.

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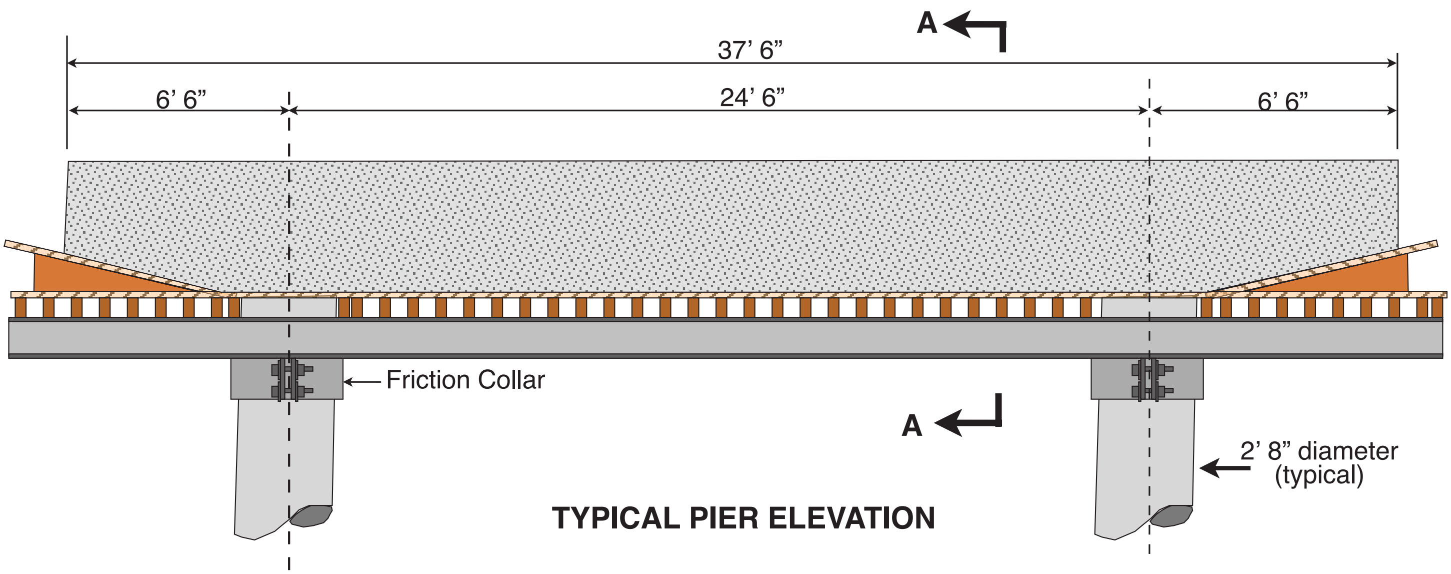
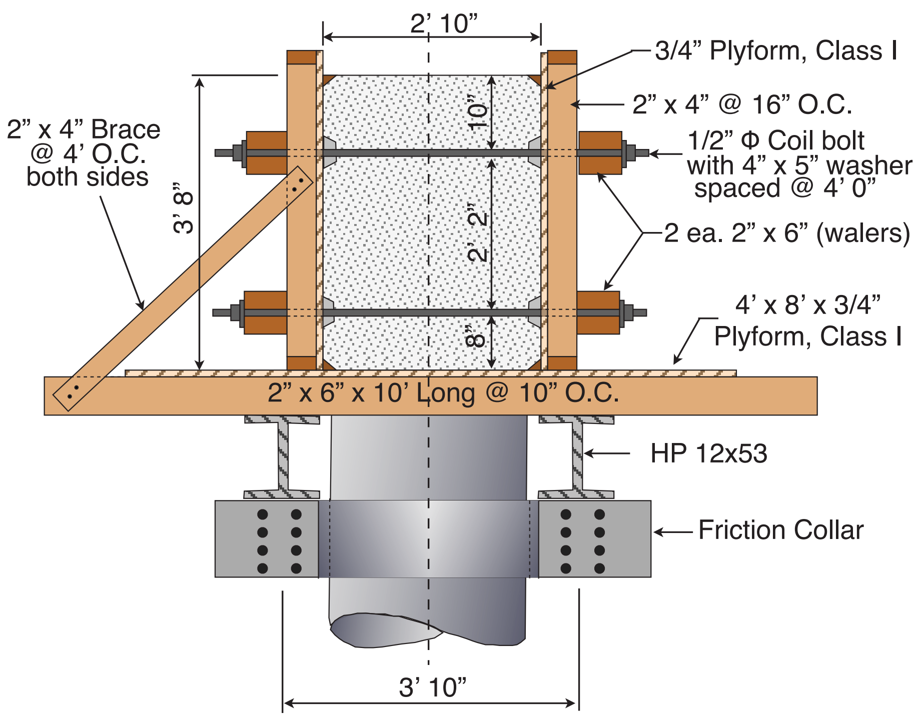
5-393.208 Examples

EXAMPLE 1—PIER CAP FALSEWORK

This is a check of the pier cap falsework details in the figure shown below. This will require the following investigations:



TYPICAL PIER ELEVATION

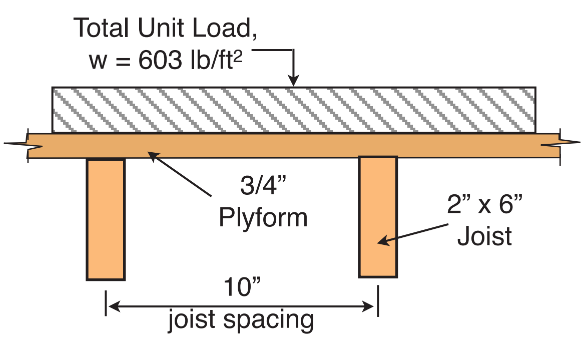


SECTION A-A

1. Plyform for bottom of pier cap
   a. Bending stress
   b. Rolling shear stress
   c. deflection
2. Joists
   a. Bending stress
   b. Horizontal shear stress
   c. deflection
3. Main supporting beams (HP 12x53)
   a. Bending stress
   b. Shear stress
   c. deflection
4. Friction collar
   a. Check against manufacturer's safe carrying capacity
   b. Required torque for collar bolts

The necessary calculations for this example items are as follows:

1.Plyform for bottom of pier cap:



Load diagram for the Plyform for bottom of pier cap.

Determine total unit applied uniform load for the plyform, w:

Concrete:	3.67 ft a 150 lb/ft^3	= 550 lb/ft^2
Plyform:	0.06 ft x 40 lb/ft^3	= 3 lb/ft^2
Live load:		= 50 lb/ft^2
Total unit load:		= 603 lb/ft^2

There are a couple of ways that the adequacy of the Plyform sheathing can be checked. First, there are graphic solutions. The charts in **Figure 5-393-200-15,** on page 5-393.200(27) can be used. The Plyform specified in the example is Class 1 and is used in the weak direction, meaning that the face grain runs parallel to the supports, joists. The chart in the upper right corner of **Figure 5-393-200-15,** on page 5-393.200(27) cover that condition. To use the chart find the total unit load, 603 lb/ft^2 on the vertical scale on the left side of the chart. Then follow that line, 603 lb/ft^2 across the chart until it intersect the line representing ¾" Plyform. Then go vertically downward to the bottom of the chart and read the maximum spacing for the joist spacing which is just a little less than 12 inches. The spacing of the joist given in the example is 10 inches, so the ¾" Plyform is adequate.

When conditions and configurations of the design proposed do not match the range offered by the charts it may be necessary to do some calculations. The following is that type of calculations. It is assumed that the Plyform is continuous over 3 or more spans and the beam formulas for continuous conditions will be used. See **Figure 5-393-200-16,** on page 5-393.200(37) for beam formulas.

a. Bending Stress:

$$f_b = \frac{M}{S}$$

where:

$M = 0.10\, wl^2$

$= 0.10 \times 603 \text{ lb/ft}^2 \times (10 \text{ in})^2 \times 1\text{ft}/12\text{in}$

$= 503$ in-lb

Section modulus: $S = 0.306 \text{ in}^3$ (ref. **Table 5-393-200-6,** on page 5-393.200(15))

$$f_b = \frac{M}{S} = \frac{503 inlb}{0.306 in^3} = 1{,}644\, psi \le 1{,}930\, psi \text{ OK!}$$

This stress is higher than the allowable stress of 1,500 psi (which would when the class of the plywood is unknown, see page 5-393.200(11). Therefore, care must be taken in determining types of Plyform used. Note, the allowable stress of 1,930 psi can only be used when it has been determined that a concrete from grade of Plyform Class I is being used. See **Table 5-393-200-7,** page 5-393.200 (15).

b. Rolling Shear Stress:

$$f_{rv} = \frac{V}{\left(\frac{I}{Q}\right)}$$ **OK!**

where:

$V = 0.6\ wL$ (ref.: **Figure 5-393-200-16,** on page 5-393.200(37))

= 0.6 x 603 lb/ft x 0.83 ft

= 300 lb

$\left(\frac{I}{Q}\right) = 4.063$ (ref.: **Table 5-393-200-6,** on page 5-393.200(15))

$$f_{rv} = \frac{300lb}{4.063in} = 73.8\,psi \approx 72\,psi$$ Close enough!

This rolling shear stress is slightly above (2.5%) the allowable rolling shear stress of 72 psi; therefore, the member is acceptable with regard to rolling shear stress with a slight overstress. See **Table 5-393-200-7,** page 5-393.200(15).

c. Deflection:

$$\Delta = \frac{0.0069wl^4}{EI}$$

(ref.: **Figure 5-393-200-16,** on page 5-393.200(37))
NOTE: Live load is not to be included in the deflection computation.

Where:

w = (603 lb/ft) – (50 lb/ft) = 553 lb/ft

l = 10 in (span or joist spacing)

E = 1,650,000 psi (ref.: **Table 5-393-200-7**)

I = 0.092 in^4/ft (ref.: **Table 5-393-200-6**)

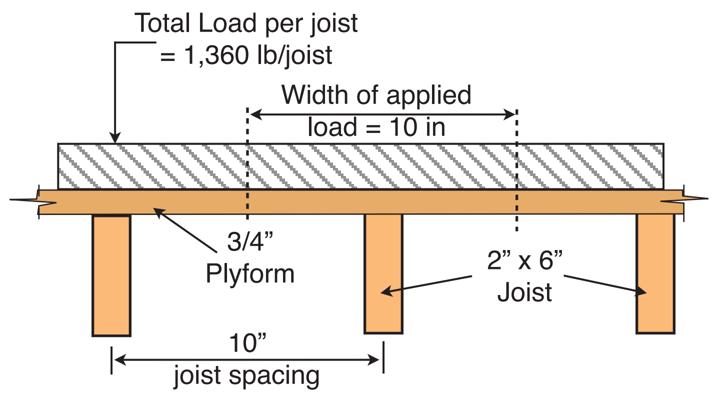
$$\Delta = \frac{0.0069 \times 553lb/ft \times (10in)^4}{1{,}650{,}000\,psi \times 0.092in^4} \times \frac{1ft}{12in}$$

$\Delta = 0.021in \le 0.037in$ OK!

The span is less than 67 inches long; therefore, the allowable deflection = 1/270 x 10 in = 0.037 in. Since actual deflection (0.021 in) is less than the allowable, (0.037 in.) the sheathing is acceptable.

2. Joist:

Determine the applied uniform unit load due to the weight of the forms and concrete per linear foot along the cap:



Load diagram for the 2" x 6" joist supporting the form bottom.

Plyform:	16 ft x 1 ft x 0.062 ft x 40 lb/ft^3 =	39.7 lb
Studs:	3.67 ft x 2 x (12 in/16 in) x 1.5 lb/ft =	8.3 lb
Plates	4 x 1 ft x 1.5 lb/ft =	6.0 lb
Walers:	8 x 1 ft x 2.3 lb/ft =	18.4 lb
	Total =	72.4 lb/lf of cap

Forms: 72.4 lb/lf of cap x (10 in/12 in) = 60 lb/joist

Concrete: 2.83 ft x 3.67 ft x 150 lb/ft3 x (10 in/12 in) = 1,298 lb/joist

Total = 1,358 lb/joist

This weight is spread over a length of 3.0 feet of each joist, for the practical purpose of these calculations it is assumed to be a uniformly distributed load over that length.

Form plus Concrete:

$$\frac{1{,}358lb/joist}{3ft} = 452.7lb/lf$$ of joist

Live load:

$$\frac{50lb}{ft^2} \times \frac{10in}{12in} = 41.7lb/lf \text{ of joist}$$

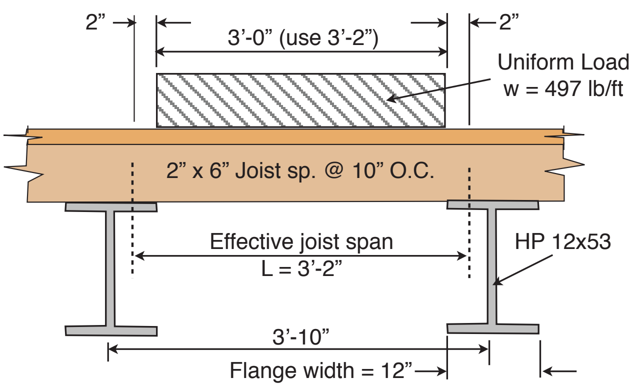
Weight of joist: 2.3 lb/lf of joist

(ref.: **Table 5-393-200-2** on page 5-393.200(9))

Total uniform load, w = 497 lb/ft

a. Bending Stress:

For beams with a very wide bearing area (such as the 12 inch wide beam flange in this example), it is reasonable to assume that the effective span begins about 2 inches back from the edge of the support. Applied to this example the effective span would be equal to (3’-10”) – 8” = 3’-2” = 3.17’**.** This assumption is considered conservative and results in a slightly higher calculated stress.



Load diagram for the 2” x 6” joist supporting the form bottom used for determining the bending moment in the joist.

The maximum bending stress in this example occurs when there is no load on the cantilevered ends of the joists. So the bending moment will be calculated as a single simple span using the following formula:

$$M = \frac{wl^2}{8}$$

$$\frac{497lb/ft \times (3.17ft)^2}{8} = 624ftlb$$

(ref.: **Figure 5-393-200-16** on page 5-393.200(37))

The Section Modulus of the 2 x 6 S4S joist is found in **Table 5-393-200-2** on page 5-393.200(9). The value of the section modulus, S = 7.56 in^3. The bending stress in the joist is calculated using the flexure formula as follows:

$$f_b = \frac{M}{S} = \frac{624ftlb}{7.56in^3} \times \frac{12in}{1ft} = 990psi \leq 1{,}065psi$$

The allowable bending stress (assuming the proposed from lumber is used material with no visible grade stamp, the allowable bending stress for red Pine will be used, $F`_b$ = 1,065 psi. This value can be found in **Table 5-393-200-3** on page 5-393.200(11).

b. Horizontal Shear Stress:

Horizontal shear stress in timber elements is calculated using the following formula, ignoring the uniform load within a distance from each support equal the depth of the member:

$$f_v = \frac{3V}{2bd}$$

where:

$$V = \frac{w(L-2d)}{2} = \frac{497\left(3.17ft - \left(2 \times 5.5\,{}^{in}\!/_{12}\right)\right)}{2} = 560lb$$

b = width of joist = 1.5 in

d = depth of joist = 5.5 in

$$f_v = \frac{3V}{2bd} = \frac{3 \times 560lb}{2 \times 1.5in \times 5.5in} 102psi \leq 175psi$$

The horizontal shear stress of 102 psi is less than the allowable shear stress for the assumed material of Red Pine, that material has an allowable shear stress of 175 psi, and therefore is adequate for horizontal shear stress, see **Table 5-393-200-3** on page 5-393.200(11).

c. Bearing stress in joist on the HP 12x53 beam:

First, determine the total vertical load on the end of each joist.

Form lumber:

$$72.4 \times \left({}^{10in}\!/_{12in}\right) = 60.3\,{}^{lb}\!/_{joist}$$

Concrete:

$$2.83ft \times 3.67ft \times 150\frac{lb}{ft^3} \times \left(\frac{10in}{12in}\right) = 1{,}298.3\,{}^{lb}\!/_{joist}$$

Live load: $$50\,{}^{lb}\!/_{ft^2} \times 8\,ft \times \left(\frac{10in}{12in}\right) = 333.3\,{}^{lb}\!/_{joist}$$

Weight of joist:

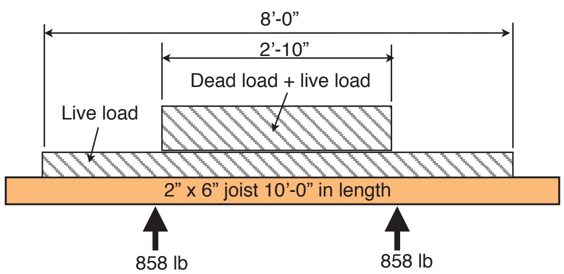
$$2.3\,{}^{lb}\!/_{ft} \times 10\,ft = 23.0\,{}^{lb}\!/_{joist}$$

Total weight per joist: 1,715 lb/joist

The bearing weight at each support:

$$\frac{1{,}715\,{}^{lb}\!/_{joist}}{2} = 858lb$$

Bearing Stress:



Load diagram for the determining the bearing stress in the joist where supported by the steel beams.

$$f_p = \frac{P}{A}$$

where:

P = 858 lb

A = 1.5 in x 12 in = 18.0 in^2

$$f_p = \frac{858lb}{18in^2} = 47.7\,psi \le 335\,psi$$

This is less than the allowable side bearing stress of 335 psi of Red Pine, see (ref.: **Table 5-393-200-3** on page 5-393.200(11)).

d. **Deflection of joist:**

Assume similar loading condition to that which causes the maximum bending stress.

$$\Delta = \frac{5wL^4}{384EI}$$

where:

w = 497 lb/ft (see page **5-393-200-(44)**)

L = 38 in

E = 1,100,000 psi (ref.: Red Pine— **Table 5-393-200-3** on page 5-393.200(11))

I = 20.80 in^4 (ref.: 2 x 6 S4S— **Table 5-393-200-2** on page 5-393.200(9))

$$\Delta = \frac{5 \times 497\,{}^{lb}\!/_{ft} \times (38in)^4}{384 \times 1{,}100{,}000\,psi \times 20.80in^4} \times \left(\frac{1\,ft}{12in}\right) = 0.049in$$

$\Delta = 0.049in \le 0.141in$ OK!

The allowable deflection = 1/270 x 38 in = 0.141 in. Since the actual deflection is less than the allowable, the member is acceptable.

3. Main Support Beam (HP12x53):

Loads will be as determined for the bearing stress in Item **2 c.** above except that the live load can be reasonably reduced to 50 psf on only the horizontal concrete surface for this member. Determine dead load on each joist which bears on the two HP12x53 beams.

Form lumber:	60.3 lb/joist
Concrete:	1,298 lb/joist
Weight of joist:	23 lb
Total Applied Dead Load:	1,381 lb/joist

Convert all of these joist loads to an equivalent uniformly distributed load on each HP12x53 beam.

Dead load:

$$\frac{1{,}381\,{}^{lb}\!/_{joist}}{2} \times \left(\frac{12in}{10in}\right) = 829\,{}^{lb}\!/_{ft}$$

Live load:

$$50\,{}^{lb}\!/_{ft^2} \times \left(\frac{2.83ft}{2}\right) = 71\,{}^{lb}\!/_{ft}$$

Weight of beam:

<u>= 53 lb/ft</u>

w = 953 lb/ft

a. Bending Stress:

$$f_b = \frac{M}{S}$$

Bending stress in a beam configured like the beam in this example must be checked at two points because there is both positive and negative bending moment. The beam extends beyond the supports creating a cantilevered beam. The bending stress must checked at mid-span between the supports, called point 2, and then at the supports, called point 1. There are no available formulas to determine these moments directly. Therefore, moments will be determined by combining two known loading conditions in the *AISC Steel Construction Manual* as follows: **Figure 5-393-200-16** on page 5-393.200(37).

$$M_1 = \frac{wL^2}{2} = \frac{953\,{}^{lb}\!/_{ft} \times (6.5\,ft)^2}{2} = 20{,}132\,ftlb$$

$$M_2 = \frac{wL^2}{8} = \frac{953\,{}^{lb}\!/_{ft} = (24.5\,ft)^2}{8} = 71{,}505\,ftlb$$

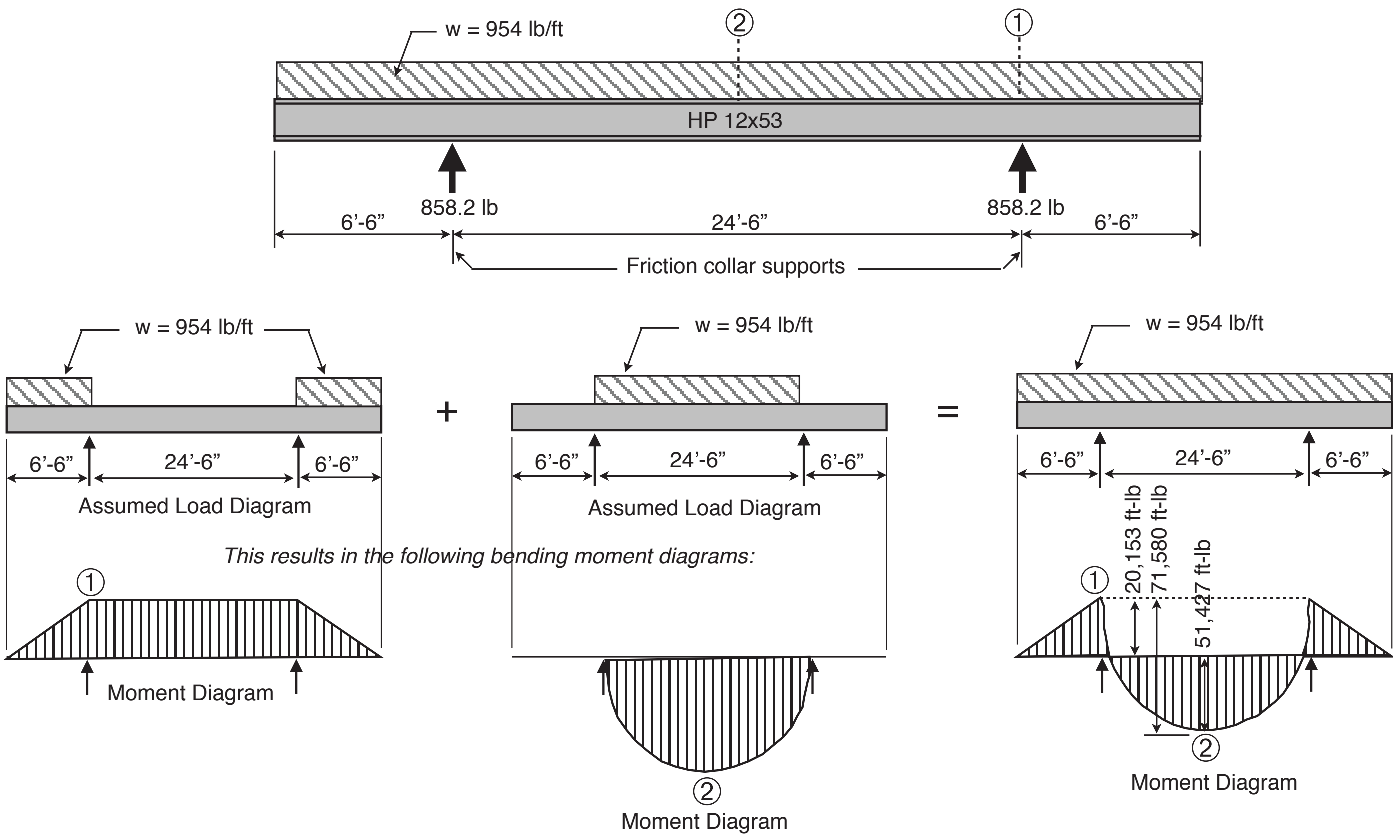
Section modulus for HP12x53 (from AISC Manual).

S = 66.8 in^3

Use bending moments from the summarized in diagram:

At location number 1:

$$f_{b1} = \frac{M_1}{S}$$



Load diagrams for the steel beams supporting the pier cap used to determine the bending moment in the beam.

$$f_{b1} = \frac{20{,}132\,ftlb \times \left(12in/_{1ft}\right)}{66.8in^3} = 3{,}617\,psi \le 25{,}000\,psi$$

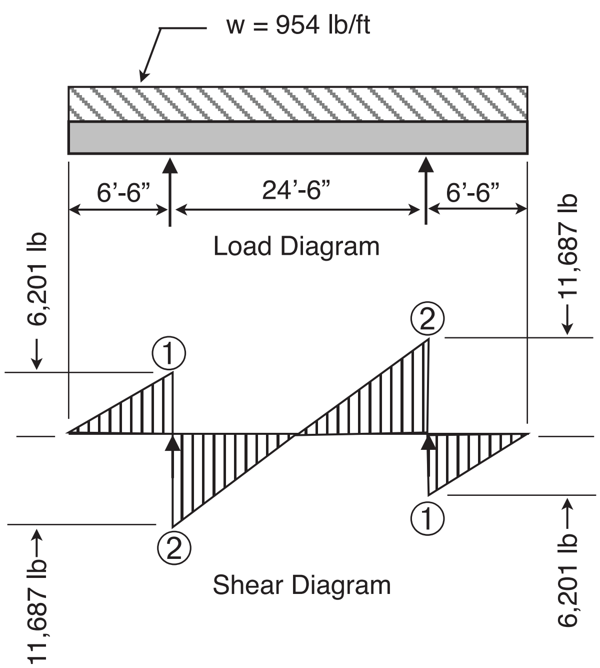
At location number 2:

$$f_{b2} = \frac{M_2}{S}$$

$$f_{b2} = \frac{51{,}373\,ftlb \times \left(12in/_{1ft}\right)}{66.8in^3} = 9{,}228\,psi \le 25{,}000\,psi$$

Assuming that the steel pile material would likely be ASTM A 36 grade, the allowable temporary bending stress is 25,000 psi (ref.: page **5-393.200(13)**). Therefore, this member qualified in bending.

b. Shear Stress in HP12x53:



Load and shear diagrams for the steel beams supporting the pier cap used to determine the shear stress in beam.

$$f_v = \frac{V}{dt_w}$$

Only the web portion of the H pile resists the vertical shear.

where:

V = vertical shear at point in question

d = depth of beam = 11.78 in

t_w = web thickness of beam = 0.435 in

Maximum shear from shear diagram at point 1:

V_1 = 6.5 ft x 953 lb/ft = 6,195 lb

Maximum shear from diagram at point 2:

V_2 = 953 lb/ft x (24.5 ft/2) = 11,674 lb

Maximum shear stress:

$$f_{v2} = \frac{V_2}{dt_w}$$

$$f_{v2} = \frac{11{,}674\,lb}{11.78in \times 0.435in} = 2{,}278\,psi \le 15{,}000\,psi$$

This is less than the allowable temporary shear stress of 15,000 psi (see page **5-393-200-(13)**).

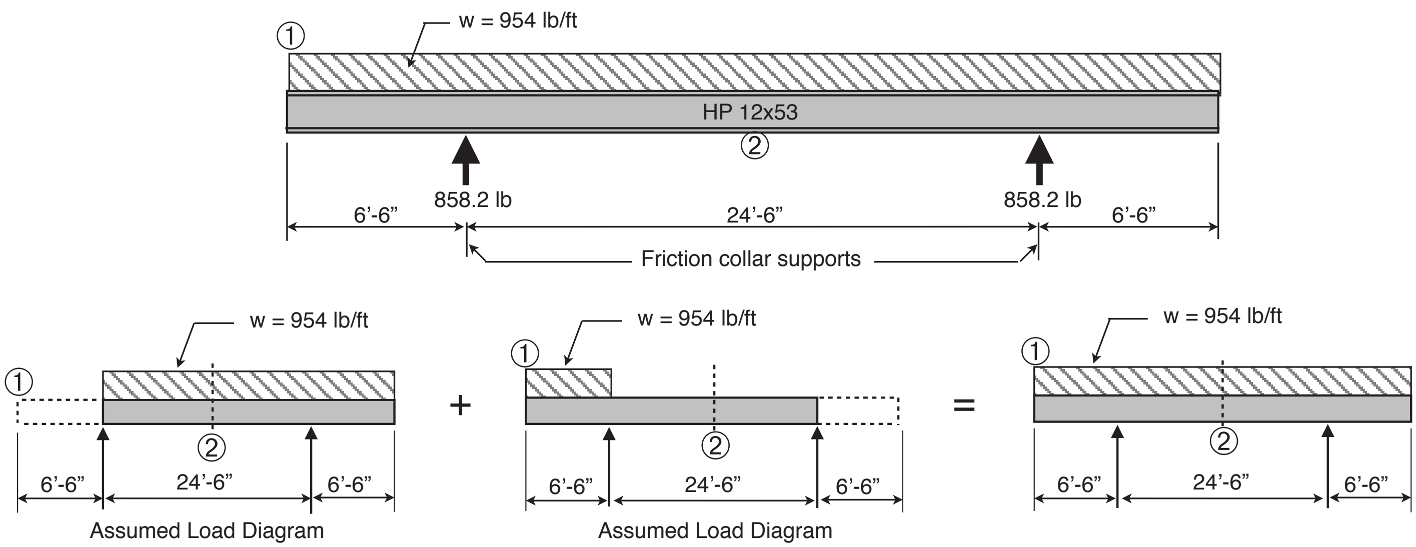
c. Deflection of HP12x53:

The loading diagram will be similar to that used for shear shown above with the exception that the live load will not be included for the deflection computations.

w = (953 lb/ft) – (71 lb/ft) = 882 lb/ft

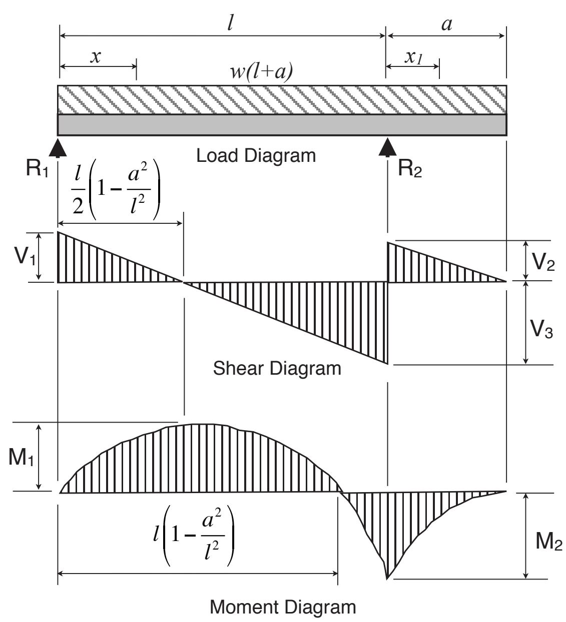
Deflection must be determined at two points for this configuration. The deflection will be calculated at the ends of the cantilever, identified as point 1, and at mid-span between the two supports, identified as point 2. There are a couple of options that can be used to calculate the required deflections. The first option would be to combine the results obtained as used above for calculating the bending moment. The other option, that will be used here, is to select a published Beam Diagram and Formulas contained in the *AISC Steel Construction Manual*, that is based on a loading condition that is representative of the problem at hand. The following is an example that could be used for determining the deflection with sufficient precision.

Since there are no readily available formulas for determining these deflection directly, this loading situation may be accommodated by combining two of the available loading diagrams in the *AISC Steel Construction Manual* (see **Figure 5-393-200-16** on page 5-393.200 (37)).



Load diagrams for the steel beams supporting the pier cap used to determine the deflection in beams.

BEAM OVERHANGING ONE SUPPORT—UNIFORMLY DISTRIBUTED LOAD



$R_1 = V_1 \ldots\ldots = \frac{w}{2l}\left(l^2 - a^2\right)$

$R_2 = V_2 + V_3 \ldots\ldots = \frac{w}{2l}\left(l + a\right)$

$V_2 \ldots\ldots = wa$

$V_3 \ldots\ldots = \frac{w}{2l}\left(l^2 + a^2\right)$

V_x (between supports) $\ldots = R_1 - wx$

V_{x1} (for overhang) $\ldots = w\left(a - x_1\right)$

M_1 (at $x = \frac{1}{2}\left[1 - \frac{a^2}{l^2}\right]$) $\ldots = \frac{w}{8l^2}\left(l + a\right)^2\left(l - a\right)$

M_2 (at R_2) $\ldots = \frac{wa^2}{2}$

M_x (between supports) $\ldots = \frac{wx}{2l}\left(l^2 - a^2 - xl\right)$

M_{x1} (for overhang) $\ldots = \frac{w}{2}\left(a - x_1\right)^2$

Δ_x (between supports) $\ldots = \frac{wx}{24EIl}\left(l^4 - 2l^2x^2 + lx^3 - 2a^2l^2 + 2a^2x^2\right)$

Δ_{x1} (for overhang) $\ldots = \frac{wx_1}{24EI}\left(4a^2l - l^3 + 6a^2x_1 - 4ax_1^2 + x_1^3\right)$

The deflection at point 2 (mid-span) is determined by the following formulas from the *AISC Manual* for the loading condition shown above.

$$\Delta_2 = \left[\frac{wx}{24EIl}\left(l^4 - 2l^2x^2 + lx^3 - 2a^2l^2 + 2a^2x^2\right)\right]$$

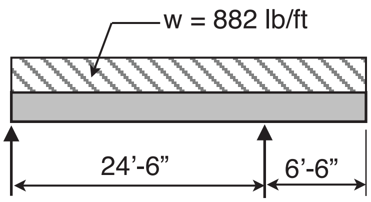
where:

w = 882 lb/ft = 73.5 lb/in

l = 24.5 ft = 294 in

x = ½ x 24.5 ft = 12.25 ft = 147 in

a = 6.5 ft = 78 in



Assumed load diagram for the steel beams supporting the pier cap used to determine the deflection in beams.

E = 29,000,000 psi

I = 393 in^4

$$\Delta_2 = \frac{73.5\,{}^{lb}\!/_{in} \times 147in}{24 \times 29{,}000{,}000\,psi \times 393in^4 \times 294in} \times$$

$$\left(294^4 - \left(2 \times 294^2 \times 147^2\right) + \left(294 \times 147^3\right) + \left(2 \times 78^2 \times 147^2\right)\right)$$

$$\Delta_2 = 0.663in \geq 0.25in$$

= 0.663 in ≥ ¼ in NOT OK!

The maximum allowable deflection in this member will be ¼ inch. See section on deflection on page **5-393-200-(36)** for further details. Since the allowable deflection at this point is exceeded, the member must either be increased in size or wedges must be placed to compensate for this deflection.

The deflection at point 1, the end of the cantilever, may be determined with sufficient accuracy by using the formula in the diagram from the *AISC Steel Construction Manual*, shown above.

$$\Delta_1 = \frac{wa}{24EI}\left(4a^2l - l^3 + 6a^3 - 4a^3 + a^3\right)$$

where:

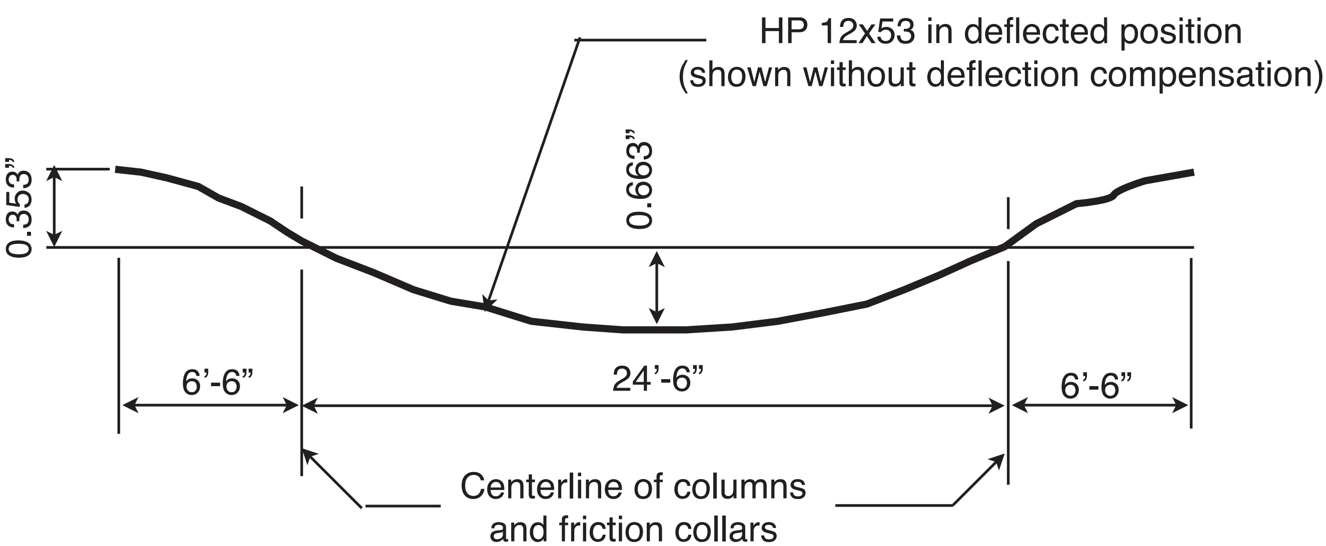
a = x_1 = 6.5 ft = 78 in

$$\Delta_1 = \frac{73.5\,{}^{lb}\!/_{in} \times 78in}{24 \times 29{,}000{,}000\,psi \times 393in^4} \times$$

$$\left(4 \times 78^2 \times 294 - 294^3 + 6 \times 78^3 - 4 \times 78^2 + 78^3\right)$$

$$\Delta_1 = -0.353in \geq 0.25in$$

Since this exceeds the allowable deflection of ¼ inches, compensation (by wedging or other means), must be made in the falsework construction, in order to obtain true lines in the completed concrete. Note: The minus sign indicates an upward deflection of the end of the HP12x53 as indicated in the diagram of the elastic curve of the structural member.



Depiction of the elastic curve of the steel beams supporting the pier cap in the deflected position.

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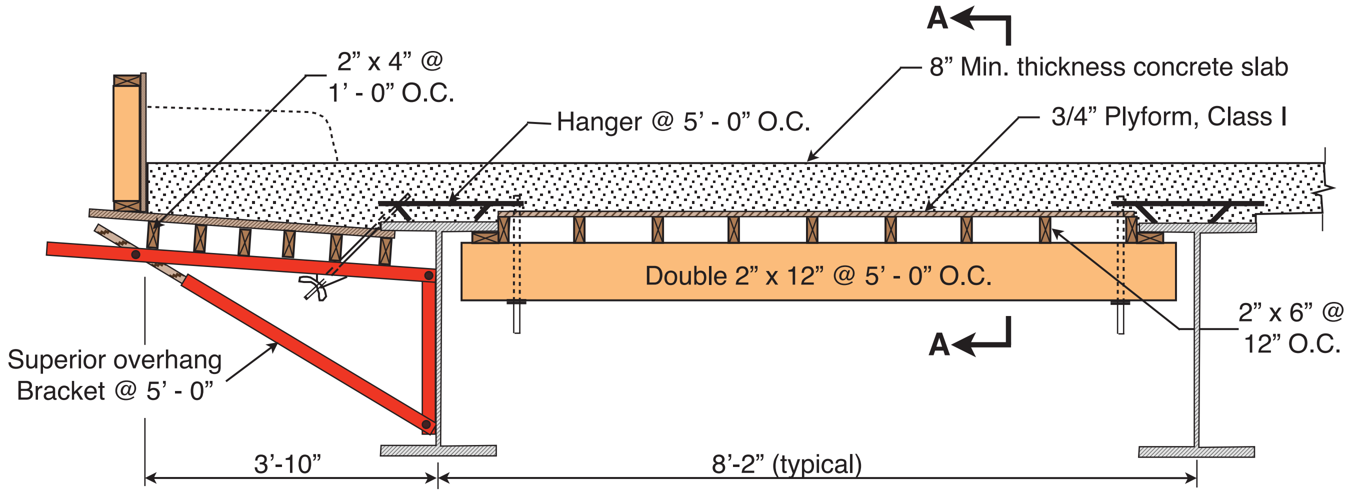
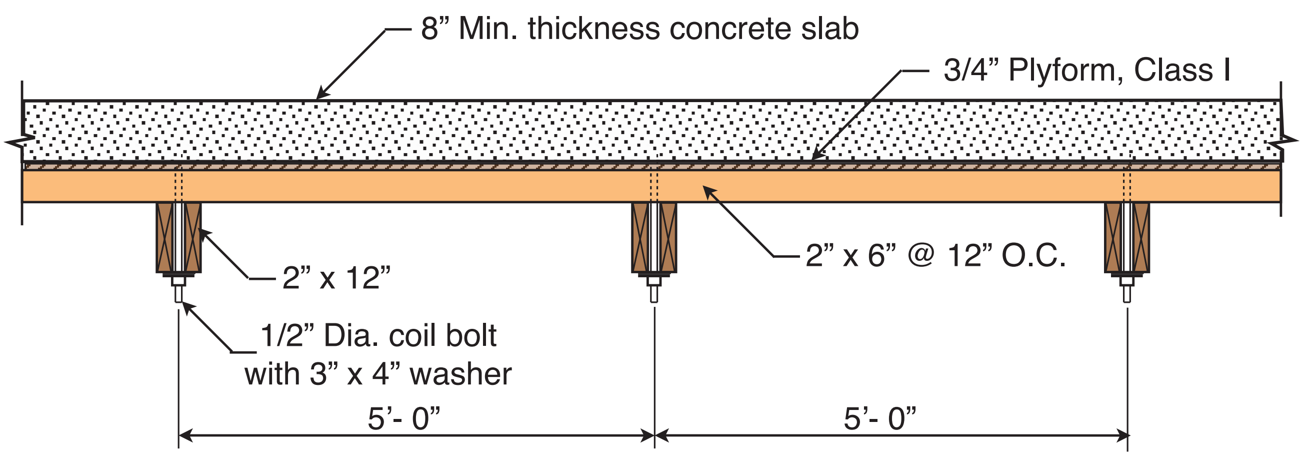
EXAMPLE 2—ROADWAY SLAB FALSEWORK

For the purpose of this example, assume the Contractor has proposed the slab falsework details shown in the diagram shown below. Assume also that the rail for the strike-off machine is supported by the fascia beams. The following investigations will then be necessary to determine the acceptability of the proposed method:



CROSS SECTION OF DECK FALSEWORK

Construction details given for Problem No. 2.



SECTION A-A

All lumber to be Douglas Fir, No. 1

Construction details given for Problem No. 2.

Interior Bays

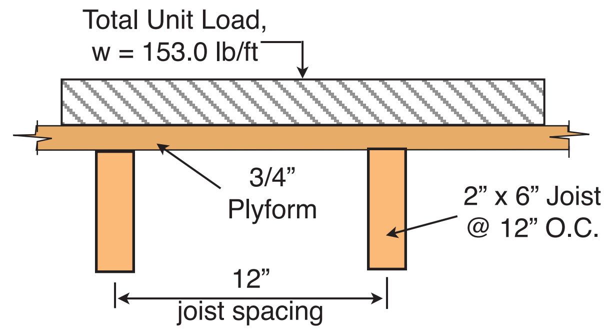
1. Plywood Sheathing
   a. Bending stress
   b. Rolling shear stress
   c. Deflection
2. Stringers
   a. Bending stress
   b. Horizontal shear stress
   c. Bearing stress
   d. Deflection
3. Joists (double 2”x12” member)
   a. Bending stress
   b. Horizontal shear stress
   c. Bearing stress from washer
   d. Deflection

Slab Overhang Falsework

1. Plywood Sheathing
   a. Bending stress
   b. Rolling shear stress
   c. Deflection
2. Stringers
   a. Bending stress
   b. Horizontal shear stress
   c. Bearing stress
   d. Deflection
3. Steel overhang bracket
   a. Safe load
   b. Deflection
4. Hanger
   a. Direct tension on bolt
   b. Capacity of hanger

INTERIOR BAYS

1. Plywood Sheathing:



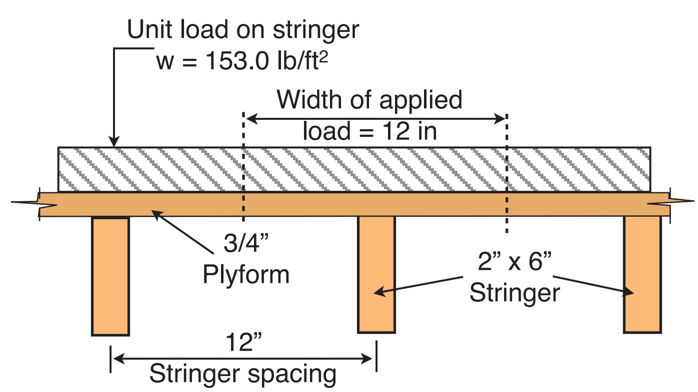
Load diagram for the Plyform for bottom of roadway slab falsework.

Determine the applied unit load, *w*:

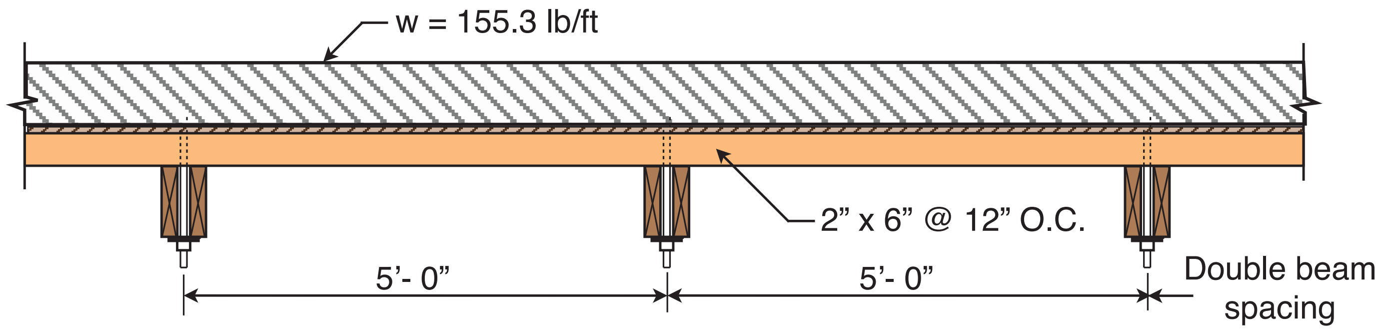
Concrete:	0.67 ft x 1 ft x 150 lb/ft^3	= 100.5 lb/ft
Plywood:	0.06 ft x 1 ft x 40 lb/ft^3	= 2.5 lb/ft
Live load:		= 50 lb/ft
	w	= 153.0 lb/ft

The requirements to evaluate items a, b, and c— Bending, Rolling Shear, and Deflection for the plywood sheathing can be checked using two different design aids contained in this manual. Both design aids automatically check all three of the criteria listed. The first method uses the Charts contained in **Figure 5-393-200-15** on page 5-393.200(27). According to the conditions of this example, 12 inch support spacing for ¾ inch Plyform Class I will safely support about 550 pounds per square-foot, psf, using the chart in the upper right corner of **Figure 5-393-200-15**. Additionally, **Table 5-393-200-4** (page **5-393-200-(14)**), can also be used to check the structural adequacy of the sheathing used in this example.

2. Stringers:



Load diagram for the 2” x 6” stringer used to idealize the applied unit load.



Load diagram for the 2" x 6" stringer used to determine the bending moment in the stringers.

Determine the applied load, *w* per foot of stringer.

Concrete, plywood and live load:	= 153.0 lb/ft
Weight of member (2x6 S4S)	= 2.3 lb/ft
Total, *w*	= 155.3 lb/ft

a. Bending Stress: The members used for this application are normally 16 feet in length that span over 3 or 4 supports, therefore we will use the following formula from **Figure 5-393-200-16** on page 5-393.200(37).

Bending Moment:

$M = 0.10wL^2$

where:

w = 155.3 lb/ft

L = 5 ft

$M = 0.10 \times 155.3 \text{ lb} \times (5.0 \text{ ft})^2 = 388$ ft-lb

Section Modulus, S = 7.56 in^3

Bending Stress:

$$f_b = \frac{M}{S} = \frac{388\,ftlb}{7.56in^3} \times \left(\frac{12in}{1ft}\right) = 616\,psi$$

f_b = 616 psi ≤ 1,375 psi OK!

The actual bending stress is less than the allowable stress of 1,375 psi for Douglas Fir, No. 1 material as shown in **Table 5-393-200-3** on page 5-393.200(11).

b. Horizontal Shear Stress:

Assuming the stringers are continuous over 3 supports the following formula will be used for calculating the maximum vertical shear, V. See **Figure 5-393-200-16** on page 5-393.200(37) for the formula.

$$V = \frac{5wl}{8} = \frac{5 \times 155.3\,{}^{lb}/_{ft} \times 4.08\,ft}{8} = 396lb$$

where:

$l = L - 2d$ = 5.0 ft-(2 × 5.5 in)/12 = 4.08 ft

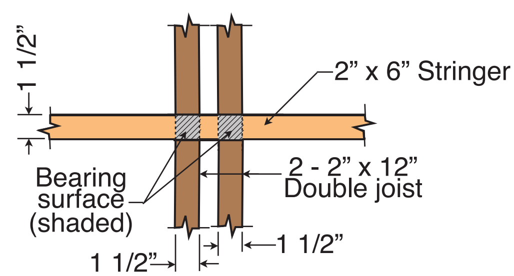
b = 1.5 in

d = 5.5 in

$$f_v = \frac{3V}{2bd} = \frac{3 \times 396lb}{2 \times 1.5in \times 5.5in} = 72\,psi \le 220\,psi \text{ OK!}$$

The allowable horizontal shear stress for Douglas Fir, No. 1 is 220 psi (ref.: **Table 5-393-200-3** on page 5-393.200 (11)**).**

c. Bearing Stress:



Detail of bearing surface of stringers on the double joist.

The maximum bearing force is from two spans continuous for the stringer, the maximum *P* will be at the center reaction point. See **Figure 5-393-200-16** on page 5-393.200(37) for the formula.

$$P = \frac{5wL}{4} = \frac{5 \times 155.3 \, {}^{lb}\!/_{ft} \times 5.0\,ft}{4} = 971\,lb$$

$$f_p = \frac{P}{A} = \frac{971\,lb}{2(1.5in \times 1.5in)} = 216\,psi \le 625\,psi \text{ OK!}$$

The temporary allowable bearing stress perpendicular to grain for Douglas Fir is 625 psi so this configuration is adequate.

Other project conditions could result in the actual bearing stresses that exceed the allowable side bearing stress for the material used. Under those conditions the allowable bearing stress can be increased by factors contained in the charts contained in **Figure 5-393-200-17** on page 5-393.200(38). In this particular example the allowable bearing stress could be increased by a factor of 1.25, which is found in the top chart on **Figure 5-393-200-17**. The allowable stress increase factor need not be figured in this example since this stress is much less than the allowable stress of 625 psi.

c. Deflection of Stringers:

The deflection is based on the assumption that the stringers are continuous over three supports. The formula is found in **Figure 5-393-200-16** on page 5-393.200(37). Additionally, the deflection is based on dead load only.

$$\Delta = \frac{wL^4}{185EI}$$

$$\Delta = \frac{105.3 \, {}^{lb}\!/_{ft} \times (5.0\,ft)^4 \times (12in)^3}{185 \times 1{,}700{,}000\,psi \times 20.80in^4} = 0.017in$$

where:

w = 155.3 lb/ft – 50 lb/ft (live load) = 105.3 lb/ft

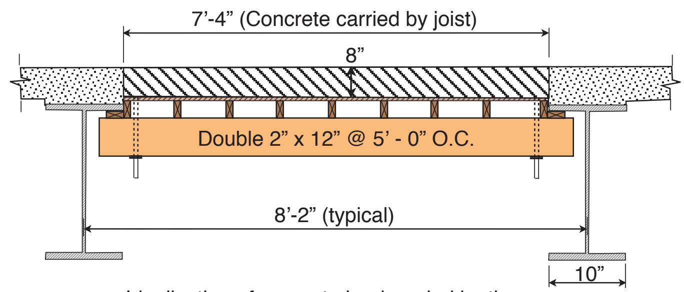
L = 5.0 ft.

E = 1,700,000 psi (ref.: **Table 5-393-200-3**)

I = 20.80 in^4 (ref.: **Table 5-393-200-2**)

The surface being formed is not exposed to view and is therefore, not subject to the normal deflection limitations. However, this value will be used later to determine the cumulative deflection of the falsework.

3. Joist (double 2 x 10 members):



Idealization of concrete load carried by the double joist.

The loads, both dead load and live load are applied to this member through eight 2 x 6 stringers. As a general rule, when the concentrated loads are applied through 3 or more crossing members, the assumption of a uniform loading may be used.

Determine uniform load on the each joist:

Concrete:

$$8in \times \left({}^{1ft}\!/_{12in} \right) \times 1ft \times 5.0\,ft \times 150 \, {}^{lb}\!/_{ft^3} = 500.0lb$$

Plyform:

0.0625 ft x 1 ft x 40 lb/ft^3 x 5.0 ft = 12.5 lb

Stringers:

8 x 2.3 lb/ft x 5.0 ft x (1/7.33) = 12.6 lb

Double 2 x 10 Joist:

2 x 3.9 lb/lf = 7.8 lb

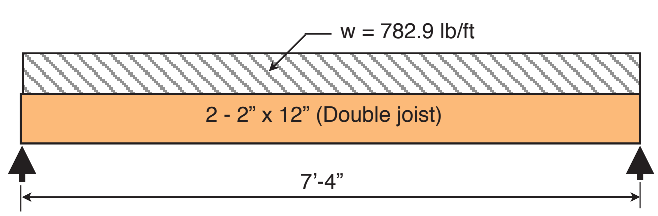
Live load:

50 lb/ft^2 x 5.0 ft x 1.0 ft = <u>250.0 lb</u>

Total, w = 782.9 lb/lf

a. Bending Stress:

First calculate the bending moment in the double joist.



Assumed load diagram for the determining the bending moment in the double joist supported at each end.

$$M = \frac{wL^2}{8} = \frac{782.9lb/lf \times (7.33ft)^2}{8} = 5{,}258\,ftlb$$

Determine the section modulus for the double 2 x 10's, either calculate the section modulus with the standard formula, or look up the value in **Table 5-393-200-2** on page 5-393.200(9).

S = 2 x 21.39 in^3 = 42.78 in^3.

Bending Stress:

$$f_b = \frac{M}{S} = \frac{5{,}258\,ftlb \times \left(\frac{12in}{1ft}\right)}{42.78in^3} = 1{,}475\,psi \approx 1{,}375\,psi$$

The allowable bending stress for Douglas Fir, No. 1 is 1,375 psi (ref.: **Table 5-393-200-3**. These members are moderately over stress by 100 psi or about 7%. This is within a reasonable tolerance. The Contractor should be notified of this situation.

b. Horizontal Shear Stress:

The formula for determining horizontal shear stress in rectangular timber members is as follows:

$$f_v = \frac{3V}{2bd}$$

where:

$$V = \frac{w(L-2d)}{2}$$

$$V = \frac{782.9lb/ft \times \left(7.33ft - \left(2 \times 9.25in \times \left(\frac{1ft}{12in}\right)\right)\right)}{2} = 2{,}266lb$$

V = 2,266 lb

b = 1.5 in

d = 9.25 in

Horizontal Shear Stress:

$$f_v = \frac{3V}{2bd}$$

$$f_v = \frac{3 \times 2{,}266lb}{2 \times 2 \times 1.5in \times 9.25in} = 122.5\,psi \le 220\,psi$$

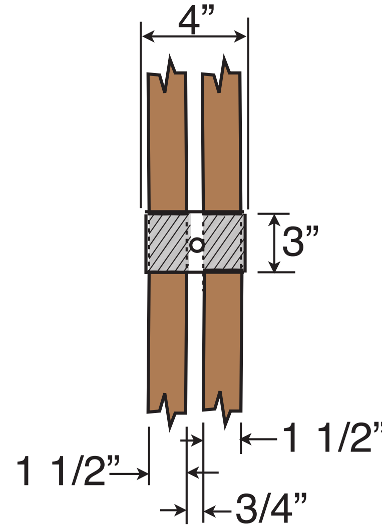
The allowable horizontal shear stress for Douglas Fir, No. 1 is 220 psi (ref.: **Table 5-393-200-3**) therefore, the double 2 x 10 joist is adequate for horizontal shear.

c. Bearing Stress on Washer:

First, calculate the bearing force P, on the washer.

$$P = \frac{782.9lb/ft \times 7.33ft}{2} = 2{,}869.3lb$$

With a 3" x 4" washer placed as shown in the detail and assume a ¾ inch space is used between the 2" x 10" members the area can be obtained from **Figure 5-393-200-17** on page 5-393.200(38) using the center chart or calculate the area directly as below.



Detail of bearing surface of the 3" x 4" washer on the double joist.

A = 3 in x 3 in. = 9.0 in^2

$$f_p = \frac{P}{A} = \frac{2{,}869.3lb}{9in^2} = 318.8\,psi \le 625\,psi$$

The allowable compression perpendicular to the grain, side bearing, for Douglas Fir is 625 psi so the washer is adequate to transfer the force from the double 2 x 10 members. However, under different conditions the actual bearing stress could exceed the allowable side bearing, if that is the case the allowable bearing stress can be increased by the factor shown in the center chart in **Figure 5-393-200-17**. That factor for the conditions in this example is 1.13. This would result in an adjusted allowable stress of 1.13 x 625 psi equaling 706 psi.

d. Deflection:

The deflection is based only on the dead load supported by these members, so the live load is deducted from the total. See **Figure 5-393-200-16** on page 5-393.200(37) for the formula.

$$\Delta = \frac{5wL^4}{384EI}$$

where:

w = 782.9 lb/ft –(50 psf x 5.0 ft) = 532.9 lb/ft

L = 7' – 4" = 7.33ft

E = 1,700,000 psi (ref.: **Table 5-393-200-3**)

I = 2 x 98.93 in4 = 197.86 in4 (ref**.: Table 5-393-200-2**)

$$\Delta = \frac{5 \times 532.9 lb/ft \times (7.33 ft)^4 \times \left(12in/_{1ft}\right)^3}{384 \times 1,700,000 psi \times 197.86 in^4} = 0.103 in$$

The cumulative deflection of the falsework in the interior bays is limited to about ½ inch (see page **5-393-200-(36)**). It can be seen that the cumulative deflection of the stringers (0.017 in) plus deflection of the double joist (0.103 in) is only 0.120 inches (approximately 1/8 inch) and is, therefore acceptable.

4. Hanger Rods:

The load on each hanger rod will be equal to the bearing load on the plate washers, 2,869.3 pounds. The ½ inch diameter coil bolts for the hangers are manufactured in various strengths such as 6,000 pound capacity, 9,000 pound capacity, etc. When required, the Contractor should furnish evidence of the safe capacity of the proposed coil bolts.

In addition to checking the coil bolt, the hanger must be checked for rated capacity. Most hangers are rated for the load carrying capacity for the entire hanger. The load on either side should not exceed one-half of this value.

SLAB OVERHANG FALSEWORK

1. Plywood Sheathing:

Maximum stress in the sheathing will occur adjacent to the beam, at the point where the concrete depth is a maximum. Assume the concrete stool height plus flange thickness at the maximum depth will be 3 inches. Where this value is known to be greater, use the known maximum value.

Determine uniform dead load on the sheathing based on this maximum thickness:

Concrete:

$$11in \times \left(1ft/_{12in}\right) \times 1ft \times 150 lb/ft^3 = 137.5 lb/ft$$

Plyform:

0.0625 ft x 1 ft x 40 lb/ft^2 = 2.5 lb/ft

Live Load:

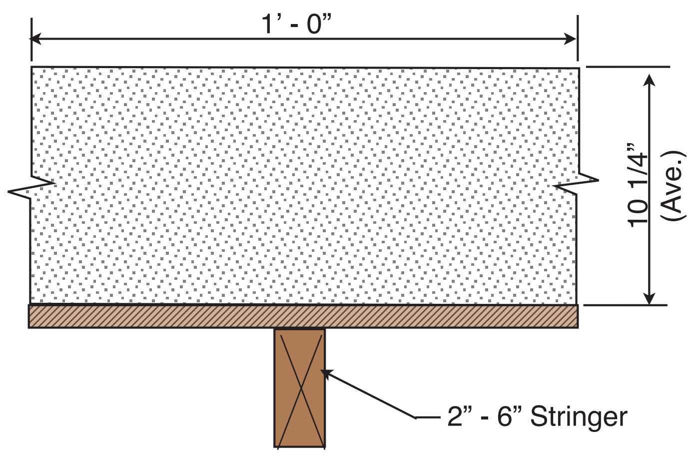
= 50.0 lb/ft

Total ,w = 190.0 lb/ft

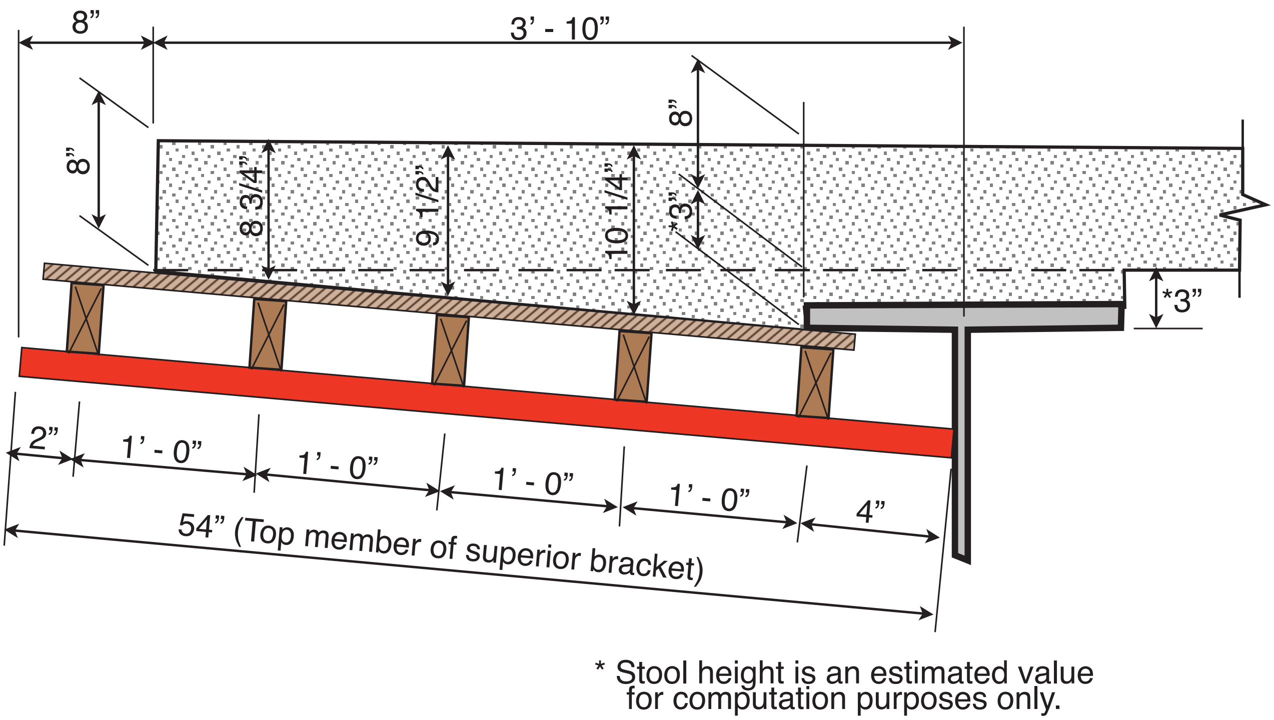
The structural adequacy of the sheathing can be checked using two design aids in this manual. First, the Chart in the upper right corner of **Figure 5-393-200-15** on page 5-393.200(27), can be used for checking the maximum uniform load for Plyform, Class I used in the weak direction for a range of spans. Based on the span length of 1' – 0", the maximum allowable uniform load is approximately 550 pounds per square-foot. Additionally, the sheathing can be checked using **Table 5-393-200-4** on page 5-393.200(14).

2. Stringer:

The second stringer from the right will be the controlling stringer for design. The first stringer only carries about one-half as much load. The average slab thickness at this controlling stringer can be determined by calculation or by scaling the drawings. In this case, an average thickness of 10 ¼ inches was scaled. The uniform load on this stringer will be:



Concrete depth for the controlling stringer in the overhang.



Details of concrete thickness at various points along the deck overhang.

Concrete:

$$10.25in \times \left(1ft/12in\right) \times 1ft \times 150lb/ft^3 = 128.1lb/ft$$

Plyform:

0.0625 ft x 1 ft x 40 lb/ft^3 = 2.5 lb/ft

Stringer:

= 2.3 lb/ft

Live Load:

= 50.0 lb/ft

Total, w = 182.9 lb/ft

The uniform load on interior stringers was 155 lb/ft. Since stringers on the overhang have the same span length as the stringers on the interior bays, their stresses may be quickly checked by ratios as follows:

a. Bending Stress:

The bending stress is calculated as follows:

$$f_b = \frac{182.9lb/ft}{155lb/ft} \times 616psi = 727psi \le 1{,}375psi$$

The actual bending stress is less than the allowable stress of 1,375 psi for Douglas Fir, No. 1 material as shown in **Table 5-393-200-3** on page 5-393.200(11).

b. Horizontal Shear Stress:

The horizontal shear stress is calculated as follows:

$$f_v = \frac{182.9lb/ft}{155lb/ft} \times 72psi = 85psi \le 220psi$$

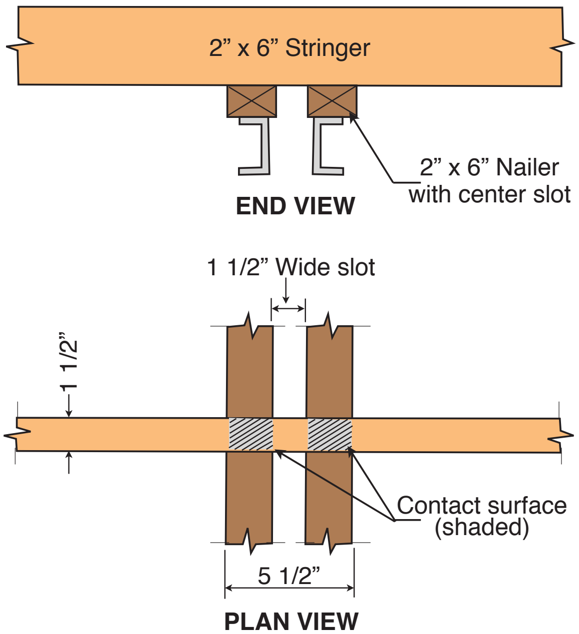
The allowable horizontal shear stress for Douglas Fir is 220 psi (ref.: **Table 5-393-200-3** on page 5-393.200 (11)).

c. Bearing Stress:

The general formula for calculating the bearing stress is as follows:

$$f_p = \frac{P}{A}$$

Using the Superior Bracket as recommended by the manufacturer with a slotted 2” x 6” top bearing surface, the bearing area, A is:



Details of the bearing surfaces of the stringers supported by the top member of a Superior bracket.

A = (5 ½" – 1 ½") x 1 ½" = 6.0 in^2

$$P = \frac{182.9 lb/ft}{155 lb/ft} \times 969 lb = 1{,}143 lb$$

$$f_p = \frac{1{,}143 lb}{6.0 in^2} = 191 psi \leq 625 psi$$

The temporary allowable bearing stress perpendicular to grain for Douglas Fir is 625 psi so this configuration is adequate.

Other project conditions could result in actual bearing stresses that exceed the allowable side bearing stress for the material used. Under those condition the allowable bearing stress can be increased by factors contained in the charts contained in **Figure 5-393-200-17** on page 5-393.200(38). In this particular example the allowable bearing stress could be increased by a factor of 1.19, which is found in the top chart on **Figure 5-393-20-17**. The allowable stress increase factor need not be figured in this example since this stress is much less than the allowable stress of 625 psi.

d. Deflection of Stringers:

The uniform load is the only factor that differs from the calculation of the interior stringers. For this member, w = 182.9 lb/ft – 50.0 lb/ft(live load) = 132.0 lb/ft. Deflection of the overhang can be determined by using ratio of the uniform loads.

$$\Delta = \frac{132.9 lb/ft}{105 lb/ft} \times 0.017 in = 0.0215 in$$

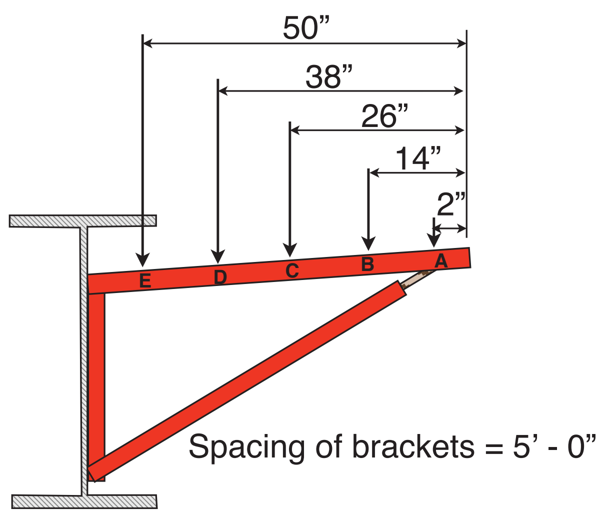
Since this surface is considered to be exposed to view, and the span length is less than 67 inches, the maximum allowable deflection will be:

$$L/270 = \frac{5.0 ft \times \left(12 in / 1 ft\right)}{270} = 0.222 in \geq 0.215 in \text{ OK!}$$

The actual deflection, 0.022 inches is less than $L/270$ of the span length and is acceptable.

3. Steel Overhang Bracket:

Superior brackets may be checked using the influence lines in **Figure 5-393-200-12** on page 5-393.200(23). To use this chart, the load on individual stringers must be determined and the distance from the outboard end of the bracket to each stringer must be determined. The following calculations are based on the diagrams below.



Detail showing the location of loads on the overhang bracket.

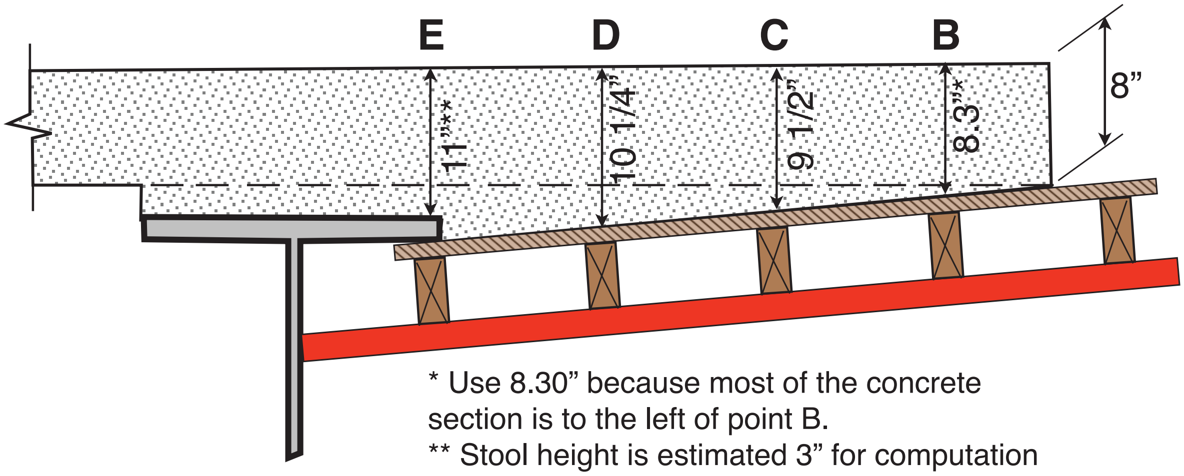
Calculation of uniform loads for each stringer:

Plyform:

$$\left(3/4 in\right) \times \left(1 ft / 12 in\right) \times 1 ft \times 40 lb/ft^2 = 2.5 lb/ft$$

Stringer load (2" x 6" S4S):

= 2.3 lb/ft



* Use 8.30” because most of the concrete section is to the left of point B.
** Stool height is estimated 3” for computation purposes only.

Details of the concrete depth at points in the deck overhang..

Live load:

= 50.0 lb/lf

Concrete loads:

$$P_A = \left[\left(\frac{8.06in \times 1in}{144in^2}\right) \times 1.0\,ft \times 150lb/ft^3\right] = 8.4lb/ft$$

$$P_B = \left[\left(\frac{8.30in \times 12in}{144in^2}\right) \times 1.0\,ft \times 150lb/ft^3\right] = 103.8lb/ft$$

$$P_c = \left[\left(\frac{9.50in \times 12in}{144in^2}\right) \times 1.0\,ft \times 150lb/ft^3\right] = 118.8lb/ft$$

$$P_D = \left[\left(\frac{10.25in \times 12in}{144in^2}\right) \times 1.0\,ft \times 150lb/ft^3\right] = 128.1lb/ft$$

$$P_E = \left[\left(\frac{11.0in \times 6in}{144in^2}\right) \times 1.0\,ft \times 150lb/ft^3\right] = 68.8lb/ft$$

These calculations are based on using the same dead load and live load for each stringer, the greatest variant is the width and thickness of the concrete supported by each stringer.

These uniform loads are combined and used to determine the maximum reaction from each stringer on the overhang bracket. The formula for determining maximum reaction is found in **Figure 5-393-200-11** on page 5-393.200(22).

Bracket load, $P_A = \dfrac{5w_A l}{4}$

where:

l = spacing of brackets = 5.0 ft

w_A = (2.5 + 2.3 + 50.0 + 8.4) = 63.2 lb/ft

w_B = (2.5 + 2.3 + 50.0 + 103.8) = 158.6 lb/ft

w_C = (2.5 + 2.3 + 50.0 + 118.8) = 173.6 lb/ft

w_D = (2.5 + 2.3 + 50.0 + 128.1) = 182.9 lb/ft

w_D = (2.5 + 2.3 + 50.0 + 68.8) = 123.6 lb/ft

a. **Actual Bracket Loads:**

P_A = 5/4 x 63.2 x (5.0 ft) = 395.0 lb

P_B = 5/4 x 158.6 x (5.0 ft) = 991.3 lb

P_C = 5/4 x 173.6 x (5.0 ft) = 1,085.0 lb

P_D = 5/4 x 182.9 x (5.0 ft) = 1,143.1 lb

P_E = 5/4 x 123.6 x (5.0 ft) = 772.5 lb

Next, the Influence Factors are taken from the chart in **Figure 5-393-200-12** on page 5-393.200(23), based on the joist location, which is the distance from the outside end of the bracket.

Since the applied loads are less than the allowable load, the coil rod and diagonal member are acceptable with regard to strength. However, other bracket components such as the hanger assembly must also be checked for

	Load On 45° Coil Rod			Load On Diagonal Member		
Stringer Identification	Bracket Load (pounds)	Influence Factor	45° Coil Rod Load (pounds)	Bracket Load (pounds)	Influence Factor	Diagonal Load (pounds)
A	395	2.6	1,027.00	395	2.8	1,106.00
B	991.3	2.1	2,081.70	991.3	1.4	1,387.80
C	1,085.00	1.8	1,953.00	1,085.00	0.90	976.50
D	1,143.10	1.5	1,714.70	1,143.10	0.50	571.60
E	772.5	1.3	1,004.30	772.5	0.1	77.30
		TOTAL =	7,780.60		TOTAL =	4,119.10
Manufacturer's Allowable Load:			9,000.00			4,733.0*

* This load is only for overhang brackets on steel beams.

strength requirements as per manufacturer's allowable loads as published in their literature.

b. Deflection of Overhang Bracket:

The manufacturer's literature indicates that the deflection is determined by summarizing the total vertical weight on the bracket. Only the weight of the concrete need be applied since the deflection due to dead load of the falsework may assumed to have already occurred prior to the placement of the concrete.

Total weight of concrete:

$$\left(\frac{8in+11in}{2}\right)\times 3.33ft\times 5.0ft\times 150lb/ft^3\times\left({}^{1ft}\!\big/_{12in}\right)=1{,}977lb$$

Deflection of the bracket is determined by using **Figure 5-393-200-12** on page 5-393.200(23). That chart indicates that the deflection from a load of 1,977 pounds will be 3/16 inch. The cumulative deflection of the overhang may now be summarized as follows:

Deflection of sheathing: negligible

Deflection of stringer: 0.022 in

Deflection of bracket: 0.190 in

Seating of wood members (2 x 1/16 in)* 0.120 in

Total deflection at center of stringer span: 0.332 in

*Abutting faces of wood members are assumed to crush 1/16 inch when heavy load is applied. This value will be less for tightly constructed falsework. In addition, wood filler used against the web as used on prestressed concrete girder must be uniformly fitting and seated to prevent overhang deflection.

The falsework along the edge of coping should, therefore, be set about 3/8 inch above the final grade to compensate for the anticipated deflection.

5. Hanger:

a. Direct Tension on Bolt:

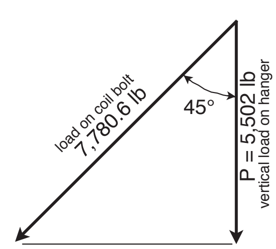
The bolt for this hanger is actually the 45° coil rod which was checked in Item 3 above. Note that the manufacturer specifies a 9,000 pounds capacity coil bolt.

Hangers are normally rated based on the vertical load carrying capacity. The vertical component of the load along the coil rod on this hanger can be determined as follows:

P = (cos45°) x 7,780.6 lb = 0.707 x 7,780.6 lb = 5,502 lb

b. Capacity of Hanger:

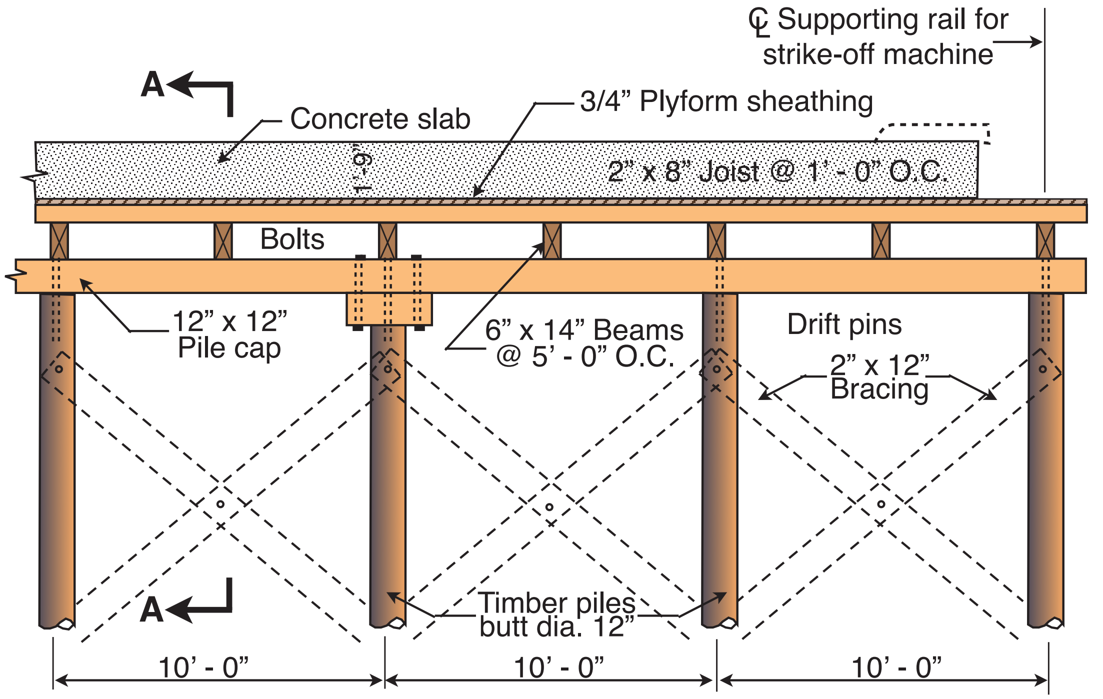
This value should not exceed ½ of the safe working load for the total hanger. Preferably, the manufacturer should furnish information as to the safe load along the 45° angle for the overhung hangers. Note: The safe working load ascribed to these hangers only applies when the device has full bearing contact on the top flange of the beam and when the hanger bolts are flush with the edge of the beam flange.



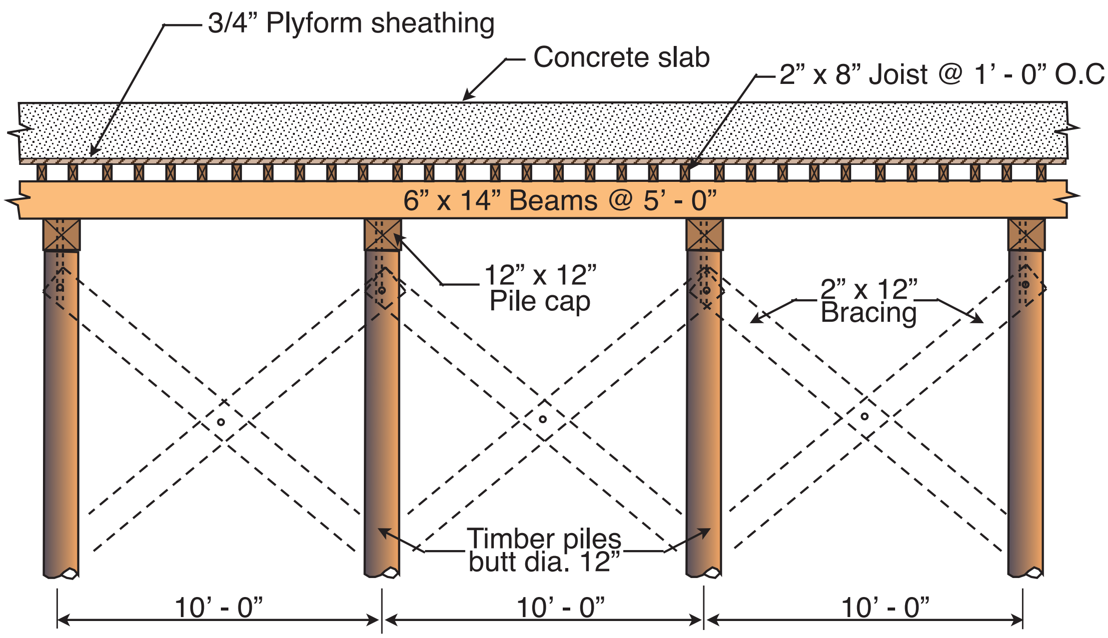
Graphic representation of equivalent vertical load on hanger.

EXAMPLE 3—SLAB SPAN FALSEWORK

For the purpose of this example, assume the Contractor has proposed the falsework scheme shown in the diagram shown below. In addition, assume they have stated that a strike-off machine weighing 8,000 pounds will be used and the strike-off rails will be located as shown in the figure (outside beam):



CROSS SECTION OF SLAB FALSEWORK



SECTION AA

The following stress investigation would be necessary:

1. Plywood Sheathing
   a. Bending stress
   b. Rolling shear stress
   c. Deflection
2. Joist (2 x 8)
   a. Bending stress
   b. Horizontal shear stress
   c. Bearing stress
   d. Deflection
3. Beams (6 x 14)
   a. Bending stress
   b. Horizontal shear stress
   c. Deflection
4. Pile Cap
   a. Bending stress
   b. Horizontal shear stress
   c. Bearing stress
   d. Deflection
5. Pile—total reaction
6. Strike-off machine support
   a. Bending stress
   b. Horizontal shear stress
   c. Bearing stress
   d. Deflection

1. Plyform Sheathing:

Calculation are as follows:

Determine applied uniform:

Concrete:

$$21in \times \left({}^{1ft}\!/\!{}_{12in} \right) \times 1ft \times 150lb/ft^3 = 262.5lb/ft$$

Plyform:

0.0625 ft x 1 ft x 40 lb/ft^2 = 2.5 lb/ft

Live Load:

= 50.0 lb/ft

Total ,w = 315.0 lb/ft

Figure 5-393-200-15 on page 5-393.200(27) indicates that even the lowest grade of Plyform (Class II) placed in the weak direction will safely support about 500 psf; therefore, the sheathing is acceptable.

2. Joist (2 x 8):

Since these members are spaced at 1' – 0", the applied uniform load is:

w = 315 lb/ft + 3.0 lb/lf (weight of the joist) = 318.0 lb/ft

a. Bending Stress:

First, calculate the bending moment in the joist.

$$M = \frac{wl^2}{8} = \frac{318.0lb/ft \times (5.0ft)^2}{8} = 993.8ftlb$$

where:

M = bending moment in joist = 993.8 ft-lb

w = uniform load = 318 lb/ft

l = span length, = 5.0 ft

Determine the section modulus of the 2 x 8 joist using **Table 5-393-200-2.**Section modulus: S = 13.14 in3

Calculate bending stress:

$$f_b = \frac{M}{S}$$

$$f_b = \frac{993.8ftlb}{13.14in^3} \times \left({}^{12in}\!/\!{}_{1ft} \right) = 907.6psi \le 1,250psi$$

The allowable bending stress for Douglas Fir, No. 2 is 1,250 psi as shown in **Table 5-393-200-3**.

b. Horizontal Shear Stress:

To determine horizontal shear stress you must first determine the vertical shear force using the following formula for two continuous spans found in **Figure 5-393-200-16** on page 5-393.200(37).

The span length used is reduced by two times the depth of the member.

$$V = \frac{5wl}{8}$$

$$V = \frac{5w\left(l-2d\right)}{8}$$

$$V = \frac{5 \times 318 lb/ft \times \left(5.0 ft - \left(2 \times 7.25 in \left(1 ft/ 12 in\right)\right)\right)}{8} = 753.6 lb$$

where:

l = 5.0 ft

w = 318.0 lb/ft

d = 7.25 in

Now calculate horizontal shear stress using the following formula:

$$f_v = \frac{3V}{2bd}$$

where:

V = 753.6 lb

d = 7.25 in

b = 1.5 in

$$f_v = \frac{3 \times 753.6 lb}{2 \times 1.5 in \times 7.25 in} = 103.9 psi \leq 220 psi$$

The allowable horizontal shear stress for Douglas Fir, No. 2 is 220 psi as shown in **Table 5-393-200-3**.

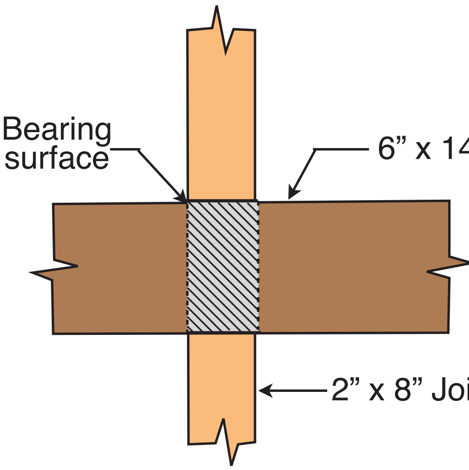
c. Bearing Stress:

Determine bearing stress of the 2 x 8 joist on the 6 x 14 beam. For two continuous spans the maximum P is at the center support. See **Figure 5-393-200-16** on page 5-393.200(37) for the formula to determine the maximum V = P.

$$P = R_2 = \frac{5wl}{4} = \frac{5 \times 318.0 lb/ft \times 5.0 ft}{4} = 1{,}987.5 lb$$

where:

w = 318.0 lb/ft



Details of of the bearing surface of the joist on the beam.

l = 5.0 ft

Calculate bearing stress using the following formula:

$$f_p = \frac{P}{A} = \frac{1{,}987.5 lb}{1.5 in \times 6.0 in} = 220.8 psi \leq 625 psi$$

where:

P = 1,987.5 lb

A = area of contact (1.5 in x 6.0 in) = 9.0 in^2

The allowable compression perpendicular to grain, side bearing, for Douglas Fir, No. 2 is 625 psi as shown in **Table 5-393-200-3** on page 5-393.200(11).

d. Deflection of 2" x 8" Joist:

Calculate the deflection using the following formula based on the dead load only:

$$\Delta = \frac{wl^4}{185EI}$$

where:

w = 318.0 lb/ft – 50 lb/ft(live load) = 268.0 lb/ft

l = 5.0 ft

E = 1,600,000 psi (ref.: **Table 5-393-200-3**)

I = 47.63 in^4 (ref.: **Table 5-393-200-2**)

$$\Delta = \frac{268 lb/ft \times (5ft)^4 \times \left(\frac{12in}{1ft}\right)^3}{185 \times 1{,}600{,}000 psi \times 47.63 in^4} = 0.021 in \leq 0.22 in$$

The limiting deflection is:

$$\frac{l}{270} = \frac{5.0 ft \times \left(\frac{12in}{1ft}\right)}{270} = 0.22 in \geq 0.021 in$$

Since the actual deflection of 0.021 inches is less than the allowable of 0.22 inches, the member is acceptable.

3. Beam (6 x 14):

Assume the Contractor has stated that these full sawn beams will be furnished in 22 foot lengths. These beams will generate two continuous spans for design purposes. See **Figure 5-393-200-16**, for the beam formulas. The first step is to calculate the applied uniform loads:

Live load, concrete, sheathing and joist:

318.0 lb/ft^2 x 5.0 ft = 1,590.0 lb/ft

Weight of 6 x 14 member (full sawn)"

6 in x 14 in x(1 ft^2/144in^2) x 40 lb/ft^3 = 23.3 lb/ft

Total = 1,613.3 lb/ft

It can be assumed that the ends of the joists will be staggered so that the critical load determined in Item 2c above will occur on any one beam.

a. Bending Stress:

First step is to calculate the bending moment in the beam using the following formula:

$$M = \frac{wl^2}{8} = \frac{1{,}613.3 lb/ft \times (10ft)^2}{8} = 20{,}166 ftlb$$

Determine the section modulus of the full sawn 6" x 14" beam using **Table 5-393-200-2**:

S = 196.0 in3 (rough cut 6 x 14)

Calculate bending stress using the following formula:

$$f_b = \frac{M}{S}$$

$$f_b = \frac{20{,}166 ftlb \times \left(\frac{12in}{1ft}\right)}{196.0 in^3} = 1{,}234.7 psi \leq 1{,}250 psi$$

The allowable bending stress is 1,250 psi; therefore, this member meets the bending strength requirements.

b. Horizontal Shear Stress for 6 x 14 Beam:

To determine horizontal shear stress you must first determine the vertical shear force using the following formula for two continuous spans found in **Figure 5-393-200-16**. The span length used is reduced by two times the depth of the member.

$$V = \frac{5wl}{8}$$

$$V = \frac{5w(l-2d)}{8}$$

$$V = \frac{5 \times 1{,}613.3 lb/ft \times \left(10ft - \left(2 \times 14\left(\frac{1ft}{12in}\right)\right)\right)}{8}$$

$$V = 7{,}730 lb$$

where:

l = 10.0 ft

w = 1,613.3 lb/ft

d = 14 in

Now calculate horizontal shear stress using the following formula:

$$f_v = \frac{3V}{2bd}$$

where:

V = 7,730 lb

d = 14 in

b = 7 in

$$f_v = \frac{3V}{2bd} = \frac{3 \times 7{,}730 lb}{2 \times 6in \times 14in} = 138.0 psi \leq 220$$

The allowable horizontal shear stress for Douglas Fir is 220 psi as shown in **Table 5-393-200-3**.

c. Bearing Stress:

Determine bearing stress of the 6 x 14 joist on the 12 x 12 pier cap. For two continuous spans the maximum P is at the center support. See **Figure 5-393-200-16** for the formula to determine the maximum V = P.

$$P = R_2 = \frac{5wl}{4} = \frac{5 \times 1{,}613.3 lb/ft \times 10 ft}{4} = 20{,}166 lb$$

where:

w = 1,613.3 lb/ft

l = 10.0 ft

Calculate bearing stress using the following formula:

$$f_p = \frac{P}{A} = \frac{20{,}166 lb}{6in \times 12.0in} = 280 psi \leq 625 psi$$

where:

P = 20,166 lb

A = area of contact (6 in x 12 in) = 72.0 in^2

The allowable compression perpendicular to grain, side bearing, for Douglas Fir is 625 psi as shown in **Table 5-393-200-3**.

d. Deflection of 6" x 14" Beam:

Calculate the deflection using the following formula based on the dead load only for two continuous spans:

$$\Delta = \frac{wl^4}{185EI}$$

where:

w = 1,613 .3 lb/ft – 50 lb/ft x 5 ft(live load) = 1,363.3 lb/ft

l = 10.0 ft

E = 1,600,000 psi (ref.: **Table 5-393-200-2**)

I = 1,372 in^4 (rough cut) (ref.: **Table 5-393-200-3**)

$$\Delta = \frac{1{,}363.3 lb/ft \times (10 ft)^4 \times \left(12in/1ft\right)^3}{185 \times 1{,}600{,}000 psi \times 1{,}372 in^4} = 0.058 in$$

$$\Delta = 0.058 in \leq 0.25 in \quad \text{OK}$$

This is less than the allowable deflection of ¼ inch for the member, but the member must also be checked later as part of the cumulative deflection.

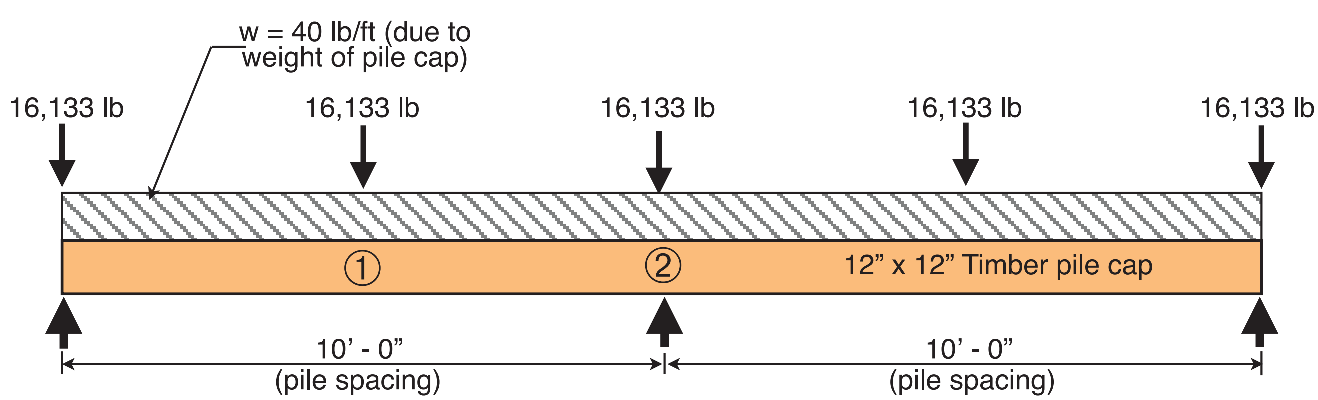
4. Pile Cap (12 x 12):

The reaction of the 6 x 14 beams on the pile cap will be determined as follows:

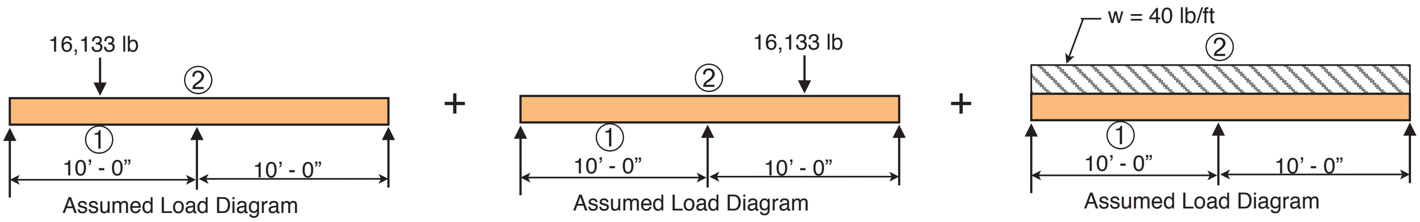
NOTE: A simple span reaction will be used since the higher reaction R_2, determined in step 3c above will occur at random locations rather than all on one cap. This simplification is also in agreement with ACI recommendations.

Live load, concrete, sheathing, joist and beam: w = 1,613.3 lb/ft

Load on pile cap: P = 1,613.3 lb/ft x 10 ft = 16,133 pounds per beam

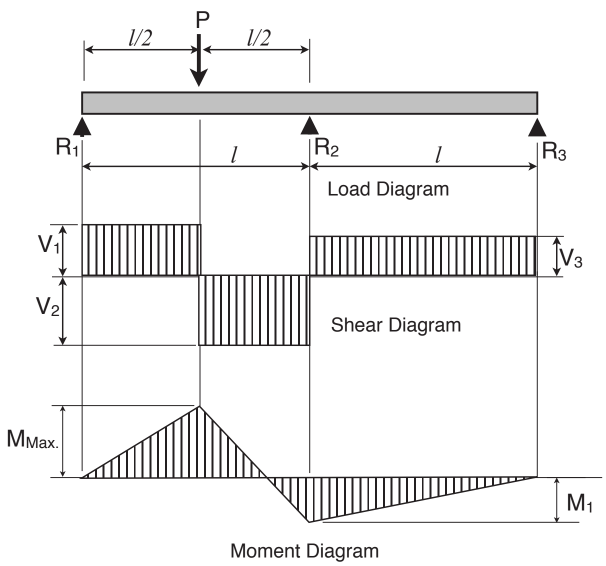


Load diagram of timber pile cap.



Three different load configurations to evaluated individually and then the results will be combined to represent the actual load condition for the pile cap.

CONTINUOUS BEAM—TWO EQUAL SPANS—CONCENTRATED AT CENTER OF ONE SPAN



Equivalent Tabular Load . . . . $= \frac{13}{8}P$

$R_1 = V_1$ . . . . $= \frac{13}{32}P$

$R_2 = V_2 + V_3$ . . . . $= \frac{11}{16}P$

$R_3 = V_3$ . . . . $= -\frac{3}{32}P$

V_2 . . . . $= \frac{19}{32}P$

$M_{max.}$ (at point of load) . . . . $= \frac{13}{64}P$

$\Delta_{Max.}$ (0.480l from R_1) . . . . . $= \frac{0.015Pl^3}{EI}$

Assume the Contractor has stated that pile caps will be furnished in 20 foot lengths. Two continuous spans will apply for design. The following loading diagram will be typical of each two span segment.

a. Bending Stress in Pile Cap:

Maximum bending stress must be checked at two points shown on the diagram below. Determine the bending moment in the cap with two different load diagrams, one from the *AISC Manual* and one that can be seen in **Figure 5-393-200-16** of this manual.

The load from the 6" x 14" beams directly over the piles are not shown since they do not cause bending in the pile cap.

$$M_1 = \left(\frac{13}{64}PL\right) - \left(\frac{1}{2}\left(\frac{3}{32}PL\right)\right) + \left(\frac{wl^2}{14.2}\right)$$

$$M_1 = \left(\frac{13}{64} \times 16{,}133lb \times 10\,ft\right) -$$

$$\left(\frac{1}{2}\left(\frac{3}{32}16{,}133lb \times 10\,ft\right)\right) +$$

$$\left(\frac{40lb/ft \times (10\,ft)}{14.2}\right) = 25{,}236lbft$$

M_1 = 25,236 ft-lb

$$M_2 = \left(2 \times \frac{3}{32}PL\right) + \frac{wl^2}{8}$$

$$M_2 = \left(2 \times \frac{3}{32}16{,}133lb \times 10\,ft\right) +$$

$$\frac{40.0lb/ft\times(10.0ft)^2}{8}=30,749lbft$$

M_2 = 30,749 lb-ft > 25,490 lb-ft

M_2 = 30,749 lb-ft Controls

Bending Stress in Pile Cap:

Calculate bending stress using the following formula:

$$f_b=\frac{M}{S}$$

$$f_b=\frac{30,749\,ftlb\times\left(12in/1ft\right)}{288in^3}=1,281psi$$

f_b = 1,281 psi > 1,250 psi Close Enough

where:

Section modulus (12 x 12 rough cut): S = 288 in^3

The allowable bending stress of Douglas Fir, No. 2 is 1,250 psi. The actual bending stress is 1,281 psi that is about 2.5 % over the allowable, which for this purpose is acceptable.

b. Horizontal Shear Stress in Pile Cap:

This stress will be the maximum over the center support. See the Beam Formulas above. First, summarize the vertical shear forces for the three diagrams used to determine bending moments.

$$V=\frac{19}{32}P+\left(\frac{1}{2}\left(\frac{3}{32}P\right)\right)+\frac{5w(L-2d)}{8}$$

$$V=\frac{19}{32}16,133lb+$$

$$\left(\frac{1}{2}\left(\frac{3}{32}16,133lb\right)\right)+$$

$$\frac{5\times40lb/ft\left(10ft-\left(2\times12in\left(1ft/12in\right)\right)\right)}{8}=10,535lb$$

Horizontal Shear Stress in Pile Cap:

Next, calculate the actual horizontal shear stress using the following formula:

$$f_v=\frac{3V}{2bd}=\frac{3\times10,535lb}{2\times12in\times12in}=109.7psi\le220psi$$

Where:

b = 12 in

d = 12 in

The allowable horizontal shear stress for Douglas Fir, No. 2 is 220 psi, see **Table 5-393-200-3**.

c. Bearing Stress in Pile Cap:

Calculate the bearing stress of the cap on the top of the piles using the following formula:

$$f_p=\frac{P}{A}=\frac{38,816lb}{113.1in^2}=343psi\le625psi$$

where:

The maximum P will be over the center support. Using the applicable formulas for reactions for the load diagrams used to determine bending moments.

$$P=R_2=2\times\frac{11}{16}P+(load\_from\_beam)+\frac{5wl}{4}$$

$$P=2\times\frac{11}{16}\times16,133lb+$$

$$16,133lb+$$

$$\frac{5\times40lb/ft\times10ft}{4}=38,816lb$$

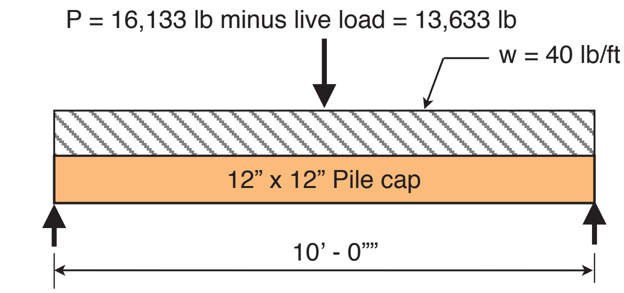
Assume a 12 inch diameter piles under the 12 x 12 cap, the contact area would be:

= 113.1 in^2

The allowable compression perpendicular to grain, side bearing, for Douglas Fir, No. 2 is 625 psi, see **Table 5-393-200-3**, this is grater than the actual of 343 psi, so the member is adequate for bearing on the piles.

d. Deflection of Pile Cap:

The exact deflection of the pile cap cannot be readily determined since a formula to cover this load situation is



Assumed load diagram for timber pile cap with reaction from 6" x 14" beam applied at mid span between two piles.

not available in the *AISC Manual*. However, formulas are available to determine an approximate value of the deflection, assuming a simple span loading condition as shown below: (NOTE: This deflection will be slightly greater than the actual deflection of the two continuous spans in the pier cap.

The deflection between piles will be:

$$\Delta = \frac{PL^3}{48EI} + \frac{5wL^4}{384EI}$$

$$\frac{13{,}633lb \times (10ft)^3 \times \left(12in/_{1ft}\right)^3}{48 \times 1{,}600{,}000psi \times 1{,}728in^4} +$$

$$\frac{5 \times 40 \times (10ft)^4 \times \left(12in/_{1ft}\right)^3}{384 \times 1{,}600{,}000psi \times 1{,}728in^4} = 0.181in$$

where:

P = Total reaction minus live load

w = 40 lb/lf

P = 16,133 lb –(50 lb/ft² x 5 ft x 10 ft) = 13,633 lb

L = 10 ft

E = 1,600,000 psi ref.: **Table 5-393-200-3**

I = 1,728 in⁴ (full sawn) ref.: **Table 5-393-200-2**

The maximum cumulative deflection of the joists, beams and pile caps will be as follows:

Joist: 0.035 in

Beams: 0.058 in

Pile cap: <u>0.181 in</u> (conservative value)

Total: 0.274 in Close enough!

It can be concluded that deflections approach a value of ¼ inch at the points of maximum deflection. Each of the individual members (joists, beam, and pile cap) are within the limited deflection value of ¼ inch and the cumulative deflection is also close enough to this value to be acceptable.

5. Pile Load:

The maximum pile load as shown in Item 4c above, P = 38,816 lb = 19.41 tons. This is not a average pile load, but rather represents the most severe conditions within the two span continuous action of the pile caps.

The average load per pile is as follows (assume each pile supports a 10 foot square area above it since the piles are spaced at 10 feet in both directions):

Sheathing, concrete and live load:

315 lb/ft² x 10 ft x 10 ft = 31,500 lb

Joists:

12 each x 10 ft x 3.0 lb/ft = 360 lb

Beams;

2 each x 10 ft x 23.3 lb/ft = 467 lb

Pile cap:

1 each x 10 ft x 40 lb/ft <u>= 400 lb</u>

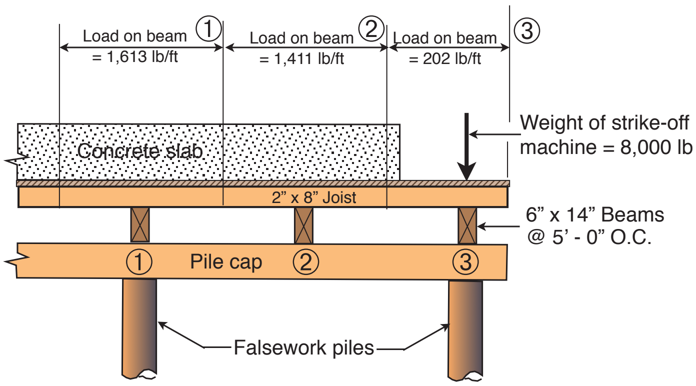
Total: = 32,727 lb = 16.36 tons

Table 5-393-200-1, indicates that piles having 10 inch diameter butts may be used for loads of up to 20 tons, 12 inch butts may be used for loads of up to 24 tons and piles having 14 inch butts may be used for loads up to 28 tons. In consideration of these values, the design proposed is well within the allowable capacities for timber piles with a minimum butt diameter of 12 inches.

6. Strike-off Machine:

Assume the Contractor (for this example) has provided information regarding the strike-off machine that indicates a total weight of 8,000 pounds. Assume also that the machine wheel-base is 5' – 0" and that the posts for supporting the strike-off rail are spaced at 5' – 0". The maximum loads on the 6 x 14 beams supporting the strike-off machine can be determined.

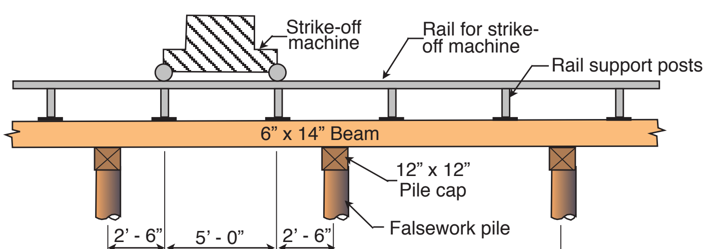
Beam No. 1 will support full design loads determined in Part 3 of this section. With the edge of the slab ending midway between beams No. 2 and No. 3, and assuming the joists are simple spans, it can be shown that beam No.



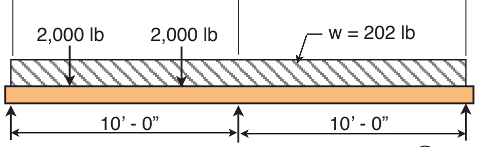
CROSS SECTION OF FALSEWORK NEAR EDGE OF SLAB

2 will carry about 7/8 of the load carried by beam No. 1, and beam No. 3 will carry about 1/8 of the load carried by beam No. 1 plus the weight of the strike-off machine.

The position of the strike-off machine shown in the load diagram will result in the maximum bending stress and maximum deflection of the 6 x 14 beam. Note that the rail support posts are placed in locations that will have approximately equal deflections. This is preferable to placing one post over the non-deflecting pier cap and having the remaining post fall at mid-span where the deflection is the greatest.



ELEVATION OF BEAM 3



ASSUMED CRITICAL LOAD FOR BEAM 3

The strike-off machine will not appreciably affect the falsework joists since the rail supports fall directly over the 6 x 14 beams. In addition, the strike-off machine will not cause bending, deflection or horizontal shear in the pile cap, since the supporting beams are placed directly over the outside rows of piles. Therefore, only the 6 x 14 beams (beam No. 3) will be investigated. To simplify calculations, this will be assumed to be a simple span rather than two spans continuous. (Use the left half of the load diagram shown.)

a. Bending Stress in 6 x 14 Beam:

First, calculation of the bending moment in the 6 x 14 beam will be done by combining the bending moment from two concentrated loads from the strike-off machine and the bending moment from the uniform load of the weight of the 6 x 14 beam.

$$M = \frac{wL^2}{8} + Pa$$

$$M = \frac{202 lb/ft \times (10 ft)^2}{8} + 2{,}000 lb \times 2.5 ft = 7{,}525 ftlb$$

Determine the section modulus of the rough cut 6 x 14 beam from **Table 5-393-200-2**.

Section modulus, S = 196.0 in^3

Calculate bending moment:

$$f_b = \frac{M}{S}$$

$$f_b = \frac{7{,}525 ftlb \times \left(12in / 1ft \right)}{196 in^3} = 461 psi \le 1{,}250 psi$$

The actual bending stress is less than the allowable of 1,250 psi.

b. Horizontal Shear Stress in 6 x 14 Beam:

First, calculation of the maximum vertical shear force, V:

$$V = 2{,}000 lb +$$

$$\frac{\left(202 lb/ft \times (10 ft) - \left(2 \times 14 in \times \left(1ft / 12in \right) \right) \right)}{2} = 2{,}774 lb$$

V = 2,774 lb

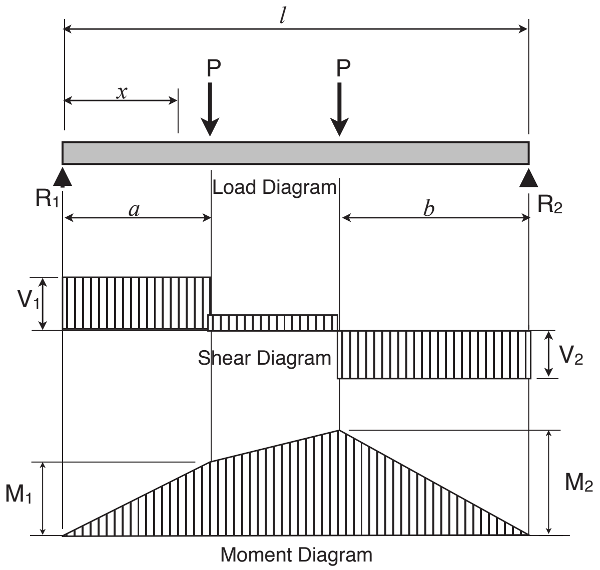
Next, calculate the horizontal shear stress:

$$f_v = \frac{3V}{2bd} = \frac{3 \times 2{,}774 lb}{2 \times 6 in \times 14 in} = 49.5 psi$$

The horizontal shear stress in the 6 x 14 beam is less than the allowable of 220 psi.

c. Bearing Stress in 6 x 14 Beam:

SIMPLE BEAM—TWO EQUAL CONCENTRATED LOADS UNSYMMETRICALLY



$R_1 = V_1$ (max. when a < b) . . . . $= \frac{P}{l}(l-a+b)$

$R_2 = V_2$ (max. when a > b) . . . . $= \frac{P}{l}(l-b+a)$

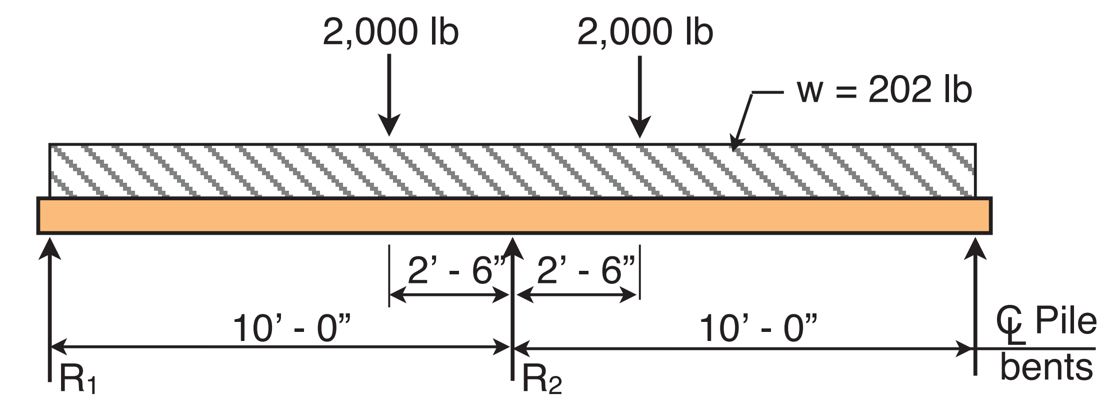
V_x (when x > a and < (l - b)) . . . . $= \frac{P}{l}(b-a)$

M_1 (max. when a > b) . . . . $= R_1 a$

M_2 (max. when a < b) . . . . $= R_2 b$

M_x (max. when x < a) . . . . $= R_1 x$

M_x (when x > a and < (l - b)) . . . . $= R_1 x - P(x-a)$



LOADING CONDITION FOR HORIZONTAL SHEAR IN BEAM ③

This critical bearing would occur with the strike-off machine centered over a pile cap. The following loading diagram will apply:

Assume two simple spans:

$$P = R_2 = 2 \times \left(\frac{Pa}{L}\right) + 2 \times \left(\frac{wL}{2}\right)$$

where:

a = 10 ft – 2' – 6" = 7' – 6" = 7.5'

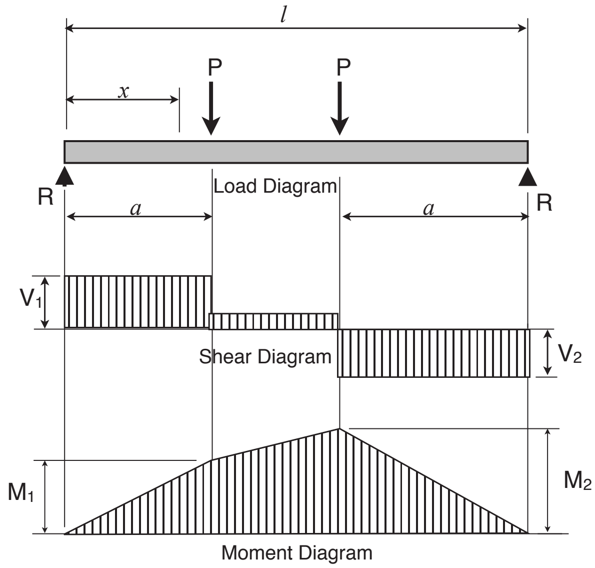
(First portion of formula from *AISC Manual*)

$$P = 2 \times \left(\frac{2{,}000 lb \times 7.5 ft}{10 ft}\right) +$$

$$2 \times \left(\frac{202 \times 10 ft}{2}\right) = 5{,}020 lb$$

Calculate bearing stress using the following formula:

SIMPLE BEAM—TWO EQUAL CONCENTRATED LOADS SYMMETRICALLY



Equivalent Tabular Load .... $= \frac{8Pa}{l}$

R = V .... $= P$

$M_{max.}$ (between loads) .... $= Pa$

M_x (when x < a) .... $= Px$

$\Delta_{max.}$ (at center) .... $= \frac{Pa}{24EI}\left(3l^2 - 4a^2\right)$

Δ_x (when x < a) .... $= \frac{Px}{6EI}\left(3la - 3a^2 - x^2\right)$

Δ_x (when x > a and < (l-a)) .... $= \frac{Pa}{6EI}\left(3lx - 3x^2 - a^2\right)$

$$f_p = \frac{P}{A} = \frac{5{,}020lb}{72in^2} = 69.7\,psi \le 625\,psi$$

where:

A = area of 6 x 14 beam on pile cap = 6 in x 14” = 72 in^2

This bearing stress is less than the allowable side bearing stress for Douglas Fir.

d. Deflection of 6 x 14 Beam under Strike-off Machine:

Assume simple span with the same loading condition used for calculation of maximum bending stress.

$$\Delta = \frac{5wl^4}{384EI} + \frac{Pa}{24EI}\left(3l^2 - 4a^2\right)$$

where:

w = 20 lb/ft

l = 10 ft

a = 2.5 ft

E = 1,600,000 psi ref.: **Table 5-393-200-3**

I = 1,372 in4 ref.: **Table 5-393-200-2**

P = 2,000 lb

$$\Delta = \frac{5 \times 202lb/ft \times \left(10ft\right)^4 \times \left(\frac{12in}{1ft}\right)^3}{384 \times 1{,}600{,}000\,psi \times 1{,}372in^4} +$$

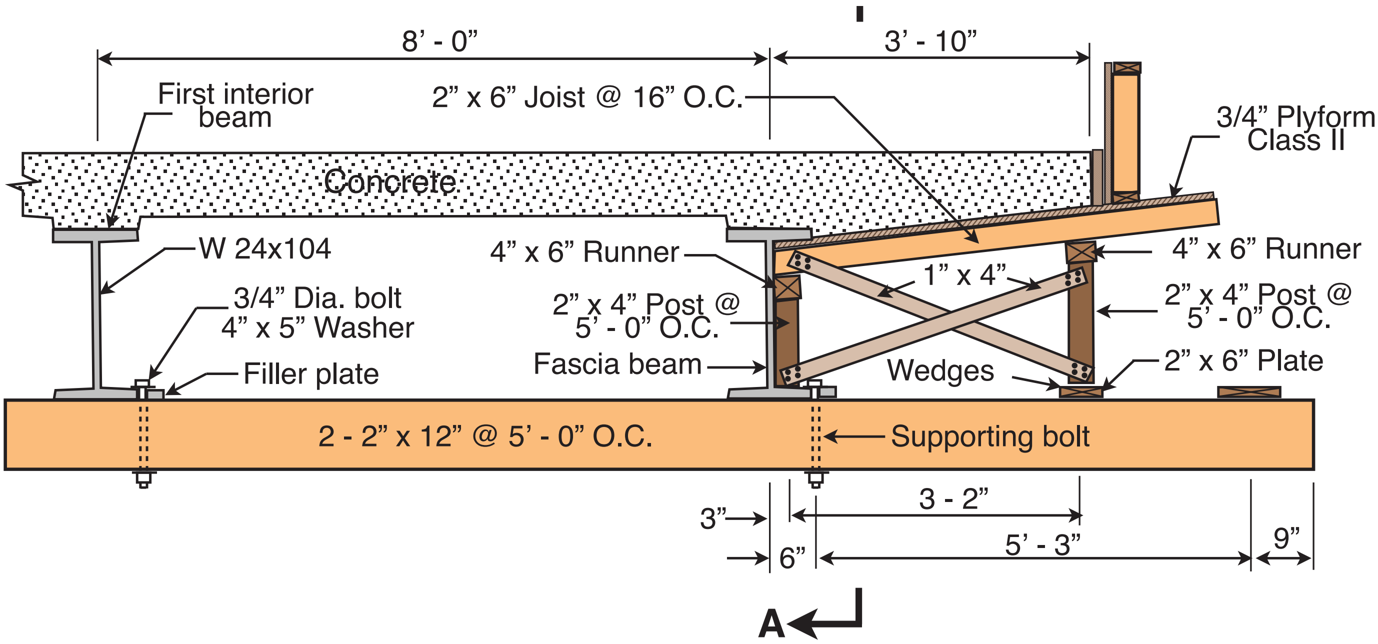
$$\frac{2000lb \times 2.5ft\left(\frac{12in}{1ft}\right)}{24 \times 1{,}600{,}000\,psi \times 1{,}372in^4}\left(3 \times \left(10ft\right)^2 - 4\left(2.5ft\right)^2\right)$$

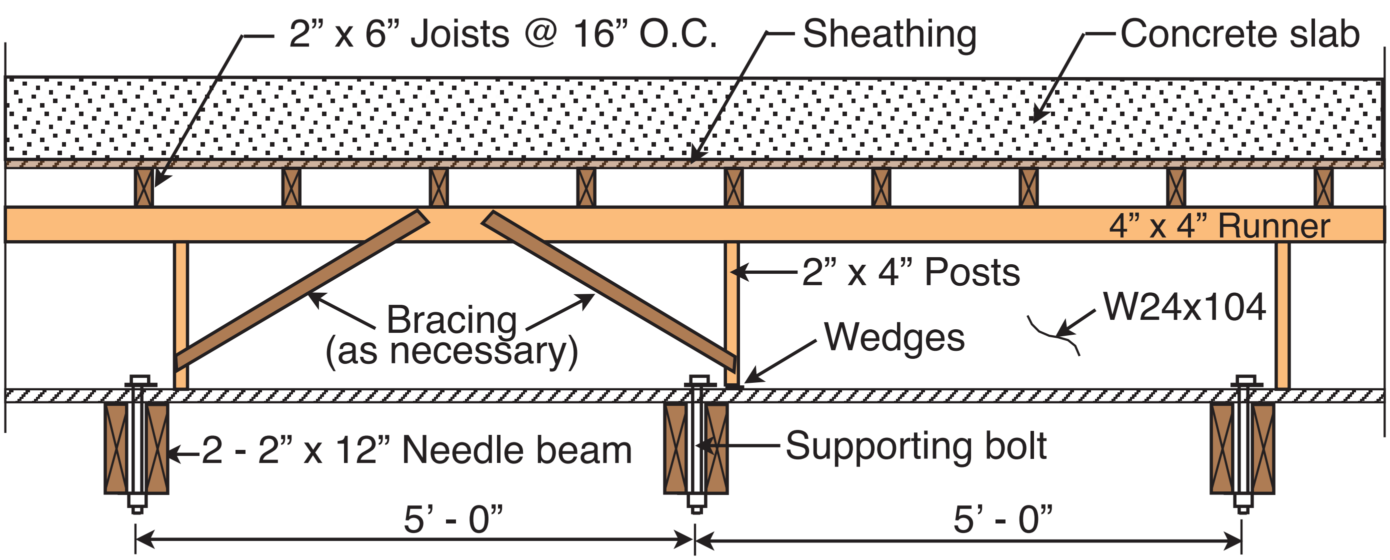
$$\Delta = 0.021in \le 1/32in$$

The calculated deflection is less than 1/32 inch and can be ignored. However, provisions should be made for the seating of wood members (about 1/16 inch per wood interface) when setting the strike-off rail to grade.

EXAMPLE NO. 4—NEEDLE BEAM

Assume that a Contractor is submitting slab falsework plans for a bridge that has shallow steel beams. Due to the difficulty of preventing rotation of the fascia beam that would occur with a cantilevered overhang bracket, they proposed a scheme that includes the needle beam falsework shown in the diagram below. Assume the strike-off machine will be run on the fascia beams. The following stressed items must be investigated:





SECTION A-A

The following stress investigation would be necessary:

1. Sheathing
    a. Bending stress
    b. Rolling shear stress
    c. Deflection
2. Joist (2 x 6)
    a. Bending stress
    b. Horizontal shear stress
    c. Bearing stress on runner
    d. Deflection
3. Runner (4 x 6)
    a. Bending stress
    b. Horizontal shear stress
    c. Bearing stress on post
    d. Deflection
4. Post (2 x 4)
    a. End bearing stress
    b. Column stress
5. Needle beams
    a. Bending stress
    b. Horizontal shear stress
    c. B earing stress on plate washer
    d. Deflection
6. Supporting bolt
    a. Tension

Calculations will be based on the assumed loading conditions shown below:

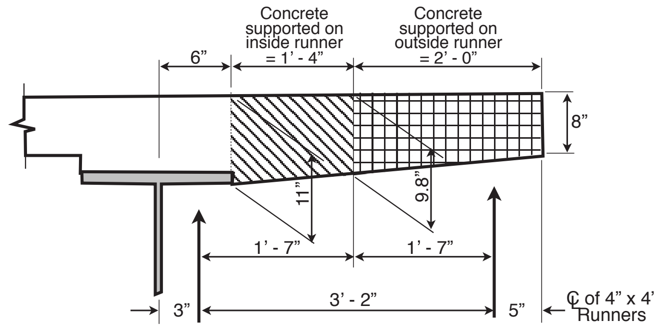
1. Sheathing:

Sheathing is supported on joists spaced at 16 inches. The maximum load on the sheathing will be near the beam flange with an estimated concrete depth of 11 inches.

Determine the uniform applied load:

Concrete:

$$150 \text{ lb/ft}^3 \times 1 \text{ ft} \times 11 \text{ in} \times (1 \text{ ft}/12 \text{ in}) = 137.5 \text{ psf}$$



Details of the assumed loading condition for the falsework supporting the overhang.

Sheathing:

$$40 \text{ lb/ft}^3 \times 1 \text{ ft} \times 0.0625 \text{ ft} = 2.5 \text{ psf}$$

Live load:

$$= \underline{50.0 \text{ psf}}$$

$$\text{Total, } w = 190.0 \text{ psf}$$

Refer to **Figure 5-393-200-15** on page 5-393.200(27), use the chart in the lower left corner. For ¾ inch thick Class II Plyform placed the strong way (which is most likely here) the safe load for 16 inch spacing is about 200 psf. The sheathing is therefore acceptable.

2. Joist (2 x 6):

Check the joists using the average slab thickness of 9.8 inches. Determine the uniform applied load:

Concrete:

$$9.8 \text{ in} \times (1 \text{ ft}/12 \text{ in}) \times 16 \text{ in} \times (1 \text{ ft}/12 \text{ in}) \times 150 \text{ lb/ft}^3 = 163.3 \text{ lb/ft}$$

Live load:

$$1.33 \text{ ft} \times 50 \text{ lb/ft}^2 = 66.7 \text{ lb/ft}$$

Sheathing:

$$40 \text{ lb/ft}^3 \times 1.33 \text{ ft} \times 0.0625 \text{ ft} = 3.3 \text{ lb/ft}$$

Joist:

$$= \underline{2.3 \text{ lb/ft}}$$

$$\text{Total, } w = 235.6 \text{ lb/ft}$$

a. Bending Stress:

First, calculate the bending moment in the joist using the formula below:

$$M = \frac{wL^2}{8} = \frac{235.6lb/ft \times (3.67ft)^2}{8} = 396.7ftlb$$

where:

L = 3' – 2" = 3.167'

Next calculate the bending stress using the formula below:

$$f_b = \frac{M}{S} = \frac{396.7ftlb \times \left(12in/_{1ft}\right)}{7.56in^3} = 630psi$$

Where:

S = section modulus = 7.56 in^3

This is less than the allowable bending stress of 1,250 psi and is, therefore, acceptable.

b. Horizontal Shear Stress:

First, calculate the maximum vertical shear force using the formula below:

$$V = \frac{w(L-2d)}{2}$$

$$V = \frac{235.6lb/ft \times \left(3.167ft - \left(2 \times 5.5in \times \left(1ft/_{12in}\right)\right)\right)}{2}$$

$$V = 265.1lb$$

where:

L = span length = 3.167 ft

d = depth of member = 3.5 in

Next, calculate the horizontal shear stress using the formula below:

$$f_v = \frac{3V}{2bd}$$

$$f_v = \frac{3 \times 265.1lb}{2 \times 1.5in \times 5.5in} = 48.2psi \le 220$$

where:

V = maximum vertical shear force = 265.1 lb

b = width of joist = 1.5 in

d = depth of joist = 5.5

The actual horizontal shear stress is less than the allowable of 220 psi and is, therefore, acceptable.

c. Bearing Stress on Runner:

Determine the reaction on the outer runner by assuming that the outer 2' – 0" of the slab concrete is supported on this runner as indicated in the previous sketch.

Concrete:

$$\left(\frac{9.9in + 8in}{2}\right) \times \left(1ft/_{12in}\right) \times 2ft \times 150lb/ft^3 = 223.8lb/ft$$

Sheathing: (for both falsework and for edge of slab form)

5.0 ft x 0.0625 ft x 40 lb/ft^3 = 12.5 lb/ft

Joist (2 x 4) for falsework and edge of slab form)

7 lf/ft x 2.3 lb/ft = 16.1 lb/ft

Runner: (4 x 6)

= 5.3 lb/ft

Live load:

50 lb/ft2 x 2 ft = <u>100.0 lb/ft</u>

Total, w = 357.7 lb/ft

Since the joists are spaced at 16 inches or 1.33 ft, the reaction per joist is:

P = 1.333 ft x 357.7 lb/ft = 476.8 lb

A = contact area = 1.5 in x 3.5 in = 5.25 in^2

Calculate bearing stress:

$$f_b = \frac{P}{A} = \frac{476.8lb}{5.25in^2} = 90.6psi \le 625psi \text{ OK!}$$

This is much less than the allowable of 625 psi and is, therefore acceptable.

d. Deflection of Joist:

Use the same loading criteria for deflection that was used for determining bending stress except that live load is deleted from the uniform load.

$$\Delta = \frac{5wl^4}{384EI}$$

where:

w = 235.6 lb/ft – 66.7 lb/ft = 168.9lb/ft

l = 3' – 2" = 3.167 ft

E = 1,600,000 psi ref.: **Table 5-393-200-3**

I = 5.36 in^4 ref.: **Table 5-393-200-2**

$$\Delta = \frac{5 \times 168.9 lb/ft \times (3.167 ft)^4 \times \left(12in/1ft\right)}{384 \times 1,600,000 psi \times 20.80 in^4}$$

$\Delta = 0.011 in \leq 0.141 in$ OK!

This surface that is exposed to view, therefore the allowable deflection for it is:

$$\frac{l}{270} = \frac{3.167 ft \times \left(12in/1ft\right)}{270} = 0.141 in$$

3. Runners (4 x 6):

Approximately 3 joists will bear on each runner span; therefore, assume the joists produce a uniform load on the runners. This uniform load has been determined in part c above.

w = 357.7 lb/ft

a. Bending Stress in Runners:

Assume the runners will be furnished in lengths of two spans or more. In following the recommended ACI design simplifications, simple span design will be used. First, determine the bending moment using the following formula.

$$M = \frac{wL^2}{8}$$

where:

w = 357.7 lb/ft

L = 5.0 ft

$$M = \frac{357.7 lb/ft \times (5 ft)^2}{8} = 1,117.8 ftlb$$

Determine the Section modulus of the (4 x 6) runner:

S = 17.65 in^3

Calculate bending stress:

$$f_b = \frac{M}{S} = \frac{1,117.8 ftlb \times \left(12in/1ft\right)}{17.65 in^3} = 760 psi$$

$f_b = 760 psi \leq 1,250 psi$ OK!

Since the bending stress in the runners is less than the allowable, therefore, the member is acceptable in bending.

b. Horizontal Shear Stress in Runners:

First, calculate the maximum vertical shear force in the runners.

$$V = \frac{w(L - 2d)}{2}$$

where:

L = 5.0 ft

b = 3.5 in

d = 5.5 in

$$V = \frac{357.7 lb/ft \times \left(5 ft - \left(2 \times 5.5 in \times \left(1ft/12in\right)\right)\right)}{2} = 730.3 lb$$

Calculate the horizontal shear stress:

$$f_v = \frac{3V}{2bd}$$

$f_v = \dfrac{3 \times 730.3 lb}{2 \times 3.5 in \times 5.5 in} = 56.9 psi \leq 220 psi$ OK!

The actual horizontal shear stress is less than the allowable shear stress, therefore the runner is acceptable for horizontal shear stress.

c. Bearing Stress From 2 x 4 Post:

Calculate the bearing stress using the following formula:

$$f_p = \frac{P}{A}$$

where:

P = 357.7 lb/ft x 5.0 ft = 1,788.5 lb

A = area of post = 1.5 in x 3.5 in

$$f_v = \frac{1{,}788.5lb}{1.5in \times 3.5in} = 340.7\,psi \le 625\,psi \quad \text{OK!}$$

The actual bearing stress, compression perpendicular to grain, is less than the allowable side bearing stress of 625 psi, therefore the runner is acceptable for bearing stress from the posts.

d. Deflection of Runner:

The deflection is based only on the dead load, therefore, the live load is subtracted from the total uniform load.

$$\Delta = \frac{5wL^4}{384EI}$$

where:

w = 357.7 lb/ft – 100 lb/ft (live load) = 257.7 lb/ft

L = span length = 5.0 ft

E = 1,600,000 psi ref.: **Table 5-393-200-3**

I = 48.53 in^4 **Table 5-393-200-2**

$$\Delta = \frac{5 \times 257.7lb/ft \times (5ft)^4 \times \left(12in/1ft\right)^3}{384 \times 1{,}600{,}000\,psi \times 48.53in^4} = 0.047in$$

$$\Delta = 0.047in \le 0.222in \quad \text{OK!}$$

Since this concrete surface will be exposed to view, therefore the allowable deflection is:

$$\frac{L}{270} = \frac{5ft \times \left(12in/1ft\right)}{270} = 0.222in$$

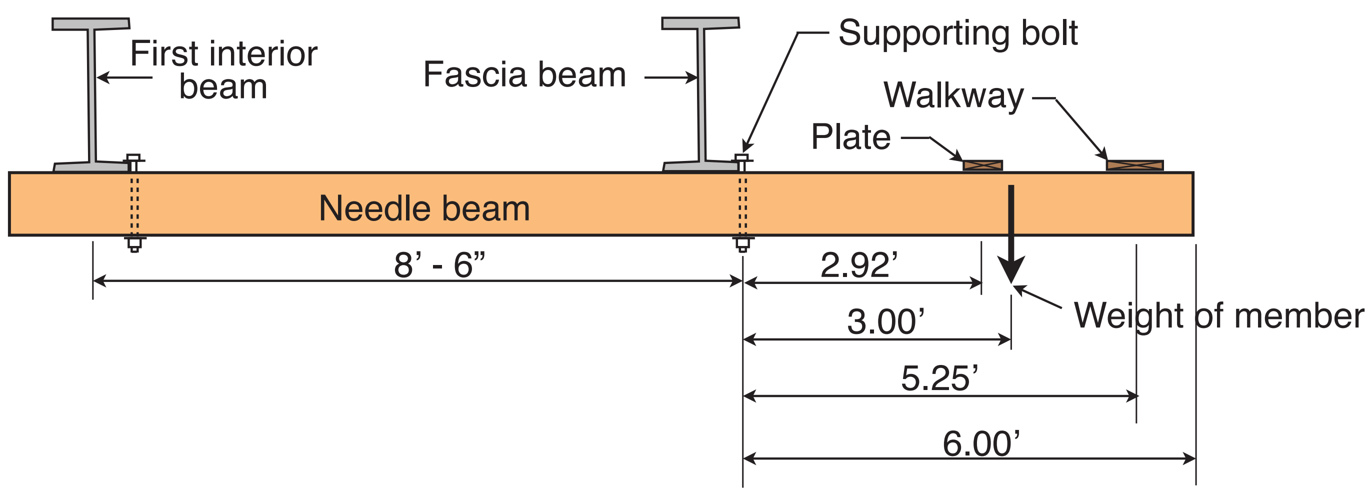
The actual deflection is less than the allowable deflection, therefore the runner acceptable.

4. 2 x 4 Posts:

The total load and resulting bearing stress was determined in Item c above (fp = 335.3 psi). By measurement on the falsework plan, the post height is determined to be 15 inches. The L/d ratio can then be determined as follows:

$$\frac{L}{d} = \frac{15in}{1.5in} = 10$$

The allowable column stress, compression parallel to grain, will be 1,700 psi as determined by **Chart 5-393-200-1** on page 5-393.200(12). Use of this Chart for this example requires two other considerations. First, the stress grade of lumber in this example is Douglas Fir, No.



Details of the location of the application of loads to the needle beam.

2, but **Chart 5-393-200-1** has data for only Douglas Fir, No. 1. The allowable End Bearing Stress, compression parallel to grain for Douglas Fir, No. 2 can be found in **Figure 5-393-200-3**, to be 1,700 psi. So, the allowable end bearing stress for columns could be interpolated based on the value for Douglas Fir, No. 1.

Secondly, the allowable end bearing stress for any column with an L/d ratio less than 15.0 is equal to the unadjusted End Bearing value.

The allowable column stress of 1,700 psi, as determined above, is greater than the actual compression parallel to grain stress (f_c = 335.3 psi), therefore the post is acceptable as a column in this example.

5. Needle Beam:

Assume each needle beam supports 5 feet of falsework. (Although the runners are continuous members they are quite flexible; therefore, simple span reactions can be safely used to determine the applied load on the needle beam.)

The loading diagram for the needle beam is as follows:

Determine unit loads on the needle beam:

Concrete, live load, sheathing, joist, and runner:

357.7 lb/ft x 5.0 ft = 1,788.5 lb

Post:

1.5 lb/ft x 1.3 ft = 2.0 lb

2 x 6 plate:

2.3 lb/ft x 5.0 ft = 11.5 lb

Total reaction at plate: = 1,802 lb

2 x 8 walk plank:

3.0 lb/ft x 5.0 lb = 15.0 lb

Live load on walkway:

0.67 sq ft x 5.0 ft x 50 psf = 167.5 lb

Total reaction at walkway: = 182.5 lb

Weight of cantilevered beam:

4.7 lb/ft x 2 x 6.0 ft = 56.4 lb

a. Bending Stress in Needle Beam:

The maximum bending moment will be at the supporting bolt. The bending moment for each load is equal to the reaction times the distance from the supporting bolt to the line of action of the load (lever arm). The calculation of the bending moment is as follows:

Reaction	x	Distance	=	Moment
1,802.0 lb	x	2.92 ft	=	5,261.8 ft-lb
182.5 lb	x	5.25 ft	=	958.1 ft-lb
56.4 lb	x	3.00 ft	=	169.2 ft-lb
2,040.9 lb	*x*		*M* =	6,389.1 ft-lb

Determine the Section Modulus of the needle beam using **Table 5-393-200-2**.

S for two 2 x 12's: $S = 31.64\ in^3 \times 2 = 63.28\ in^3$

Calculate the actual bending stress using the formula shown below:

$$f_b = \frac{M}{S}$$

$$f_b = \frac{6{,}389.1\,ftlb \times \left(12in/1ft\right)}{63.28in^3} = 1{,}211.6\,psi \le 1{,}250\,psi$$

The actual bending stress is less than the allowable bending stress for Douglas Fir, No. 2, see **Table 5-393-200-3**, therefore, the needle beam is acceptable for bending stress.

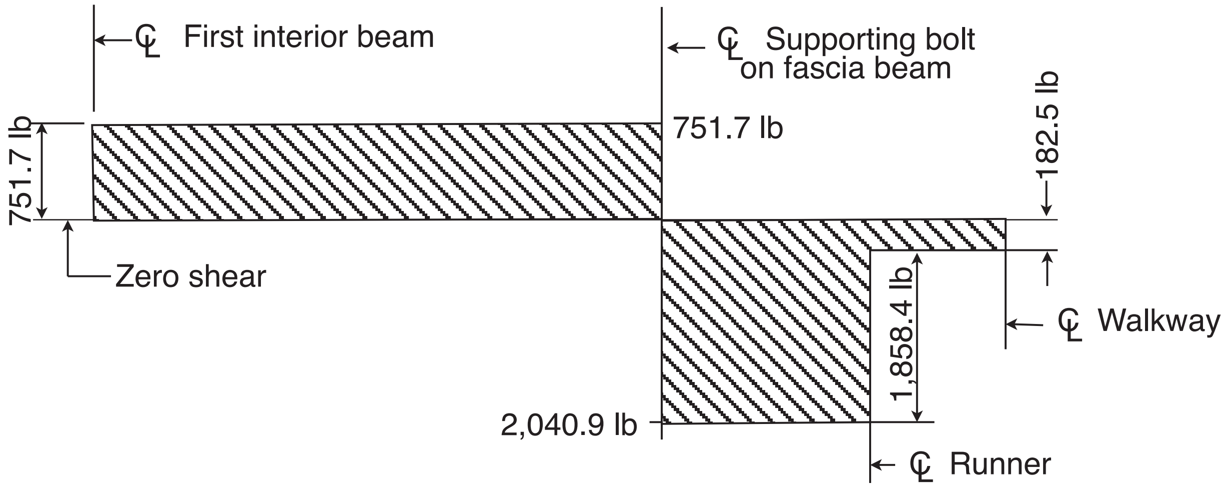
b. Horizontal Shear Stress in Needle Beam:

The shear stress in this member can be most easily visualized by drawing a shear diagram. To do this, the reaction, *P* at the first beam must be determined.

$$P = \frac{M}{L} = \frac{6{,}389.1\,ftlb}{8.5\,ft} = 751.7\,lb$$

The maximum vertical shear, *V* will be 2,038.4 pounds. Since there is no significantly large uniform load, the shear is not noticeably reduced by using the shear at a distance equal to the depth of the beam, *d* from the support.

Calculate horizontal shear stress using the formula shown below:



Shear diagram for the loaded needle beam.

$$f_v = \frac{3V}{2bd}$$

$$f_v = \frac{3 \times 2{,}040.9lb}{2 \times 2 \times 1.5in \times 11.25in} = 90.7\,psi \leq 220\,psi \leq$$

The actual horizontal shear in the needle beam is less than the allowable, therefore the member is acceptable for horizontal shear.

c. Bearing Stress on Plate Washer:

The bearing reaction, as determined from the shear diagram constructed above, will be:

P = 751.7 lb + 2,040.9 lb = 2,792.9 lb

The contact area for a 4” x 5” plate washer, as determined from the center Chart in **Figure 5-393-200-17** on page 5-393.200(38), is as follows:

A = 15.0 sq in

Calculate the bearing stress on the plate washer using the formula below:

$$f_p = \frac{P}{A} = \frac{2{,}792.6.2lb}{15in^2} = 186.2\,psi \leq 625\,psi$$

This is less than the allowable stress of 625 psi, therefore the washer size is adequate.

d. Deflection of Needle Beam:

The needle beam can be set to the plan elevation after the deflection due to the weight of the members has occurred. Therefore, the calculations for deflection must only determine the additional amount of deflection due to the weight of the concrete applied through the runner.

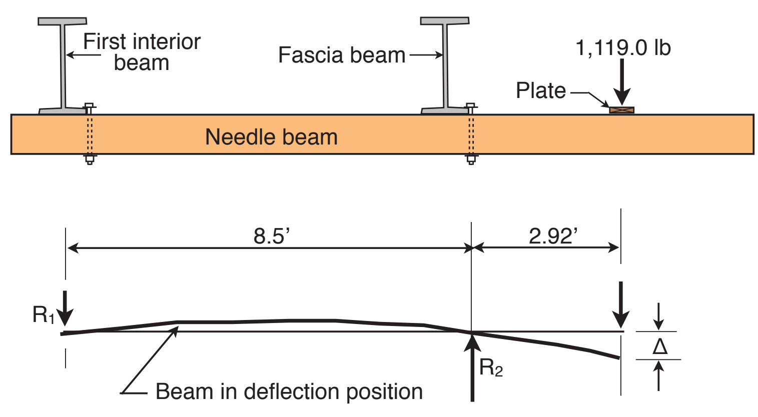
The uniform weight of the concrete on the outside runner has already been determined to be 223.8 pounds per foot. The concrete loads and reactions on each needle beam will be as follows:

P = 223.8 lb/ft x 5.0 ft = 1,119.0 lb

R_1 x 8.5 ft = 1,119.0 lb x 2.9167 ft

R_1 = -384.0 lb

R_2 = 1,119.0 + 384.0 lb

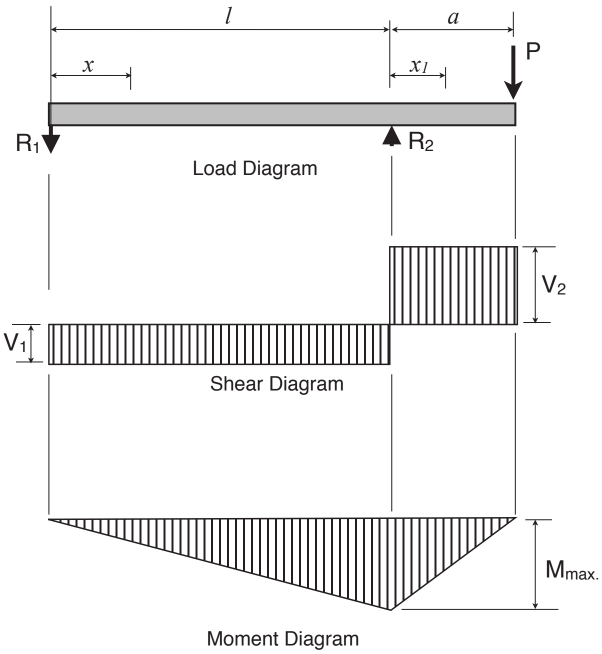


Load diagram and elastic curve of needle beam.

R_2 = 1,503.0 lb

The formula determining the deflection of the needle beam can be found in the diagram from the *AISC Manual*, as follows:

BEAM OVERHANGING ONE SUPPORT—CONCENTRATED LOAD AT END OF OVERHANG



$R_1 = V_1$ . . . . $= \frac{Pa}{l}$

$R_2 = V_1 + V_2$ . . . . $= \frac{P}{l}(l + a)$

V_2 . . . . $= P$

$M_{max.}$ (at R_2) . . . . $= Pa$

M_x (between supports) . . . . $= \frac{Pax}{l}$

M_{x1} (for overhang) . . . . $= P(a - x_1)$

$\Delta_{max.}$ (between supports at $x = \frac{l}{\sqrt{3}}$) . . . $= 0.06415\frac{PaL^2}{EI}$

$\Delta_{max.}$ (for overhang at $x_1 = a$) . . . . $= \frac{Pa^2}{3EI}(l + a)$

Δ_x (between supports) . . . . $= \frac{Pax}{6EIl}(l^2 - x^2)$

Δ_{x1} (for overhang) . . . . $= \frac{Px_1}{6EI}(2al + 3ax_1 - x_1^2)$

$$\Delta = \frac{Pa^2}{3EI}(l + a)$$

where:

P = 1,119.0 lb

L = 8.5

a = 2.9167 ft

E = 1,600,000 psi

I = (two 2 x 12s) = 2 x 177.98 in^4 = 355.96 in^4

$$\Delta = \frac{Pa^2}{3EI}(l + a)$$

$$\Delta = \frac{1{,}119.0lb \times \left(2.9167ft \times \left(12in/1ft\right)\right)^2}{3 \times 1{,}600{,}000psi \times 355.96in^4} \times$$

$$\left(8.5ft + 2.9167ft\left(12in/1ft\right)\right) = 0.11in$$

The falsework should be set 0.11 inches higher at the outer runner to compensate for deflection of the needle beam. In addition, five or more wood to wood surfaces should exist in the support falsework, all of which will tend to seat (deflection downward) when the concrete load is applied. A commonly used practice is to set the falsework "high" by 1/16 inch per interface or 5/16 inch in this example. The net height adjustment to the outer runner would them be:

0.11 in + 0.31 in = 0.42 in (above plan elevation)

EXAMPLE NO. 5—COLUMN APPLICATIONS

1. Wood Columns
2. Steel Columns

1. Wood Columns:

A Douglas Fir, No. 1, 6” x 8” S4S member will be used as a falsework column to support a load of 16,000 pounds. The unsupported length of the column is 14 feet. To determine if this member is acceptable with regard to calculated stress, the following computations are necessary:

The actual compressive stress in the member is:

$$f_c = \frac{P}{A} = \frac{16{,}000lb}{5.5in \times 7.25in} = 401.3psi$$

The allowable compressive is dependent on the l/d ratio.

$$\frac{l}{d} = \frac{14ft \times \left(12in/1ft\right)}{5.5in} = 30.55$$

From **Chart 5-393-200-1** on page 5-393.200(12), the allowable compressive stress for a Douglas Fir, No. 1, column with an *l/d* ratio of 30.55 is 550 psi. Since the actual compressive stress parallel to grain, as calculated above is 401.3 psi, which is less than the allowable stress, to column is acceptable.

2. Steel Columns:

A length of new HP 10x42 piling will be used as a falsework column to support a load of 40,000 pounds. The unsupported length of the column is 16 feet. The following calculations are necessary to determine acceptability of this column.

The actual compressive stress in the member is calculated using the following formula:

$$f_c = \frac{40{,}000lb}{12.4in^2} = 3{,}226psi$$

where:

The area of a HP 10x42 = 12.4 in^2 (ref.: *AISC Manual*)

r = 2.41 in (ref.: *AISC Manual*, use smallest value)

K = 1.0 (ref.: *AISC Manual*)

The allowable stress is determined by the appropriate formula from **page 5-393.200(13)** of this manual.

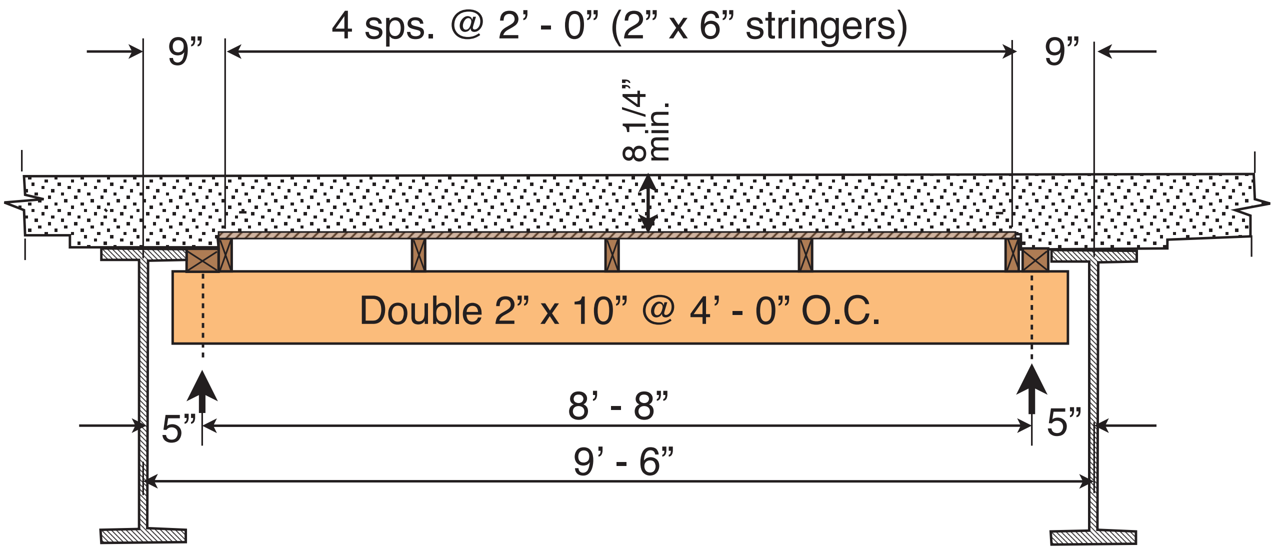
$$F_c = 16{,}980psi - 0.53 \times \left(\frac{KL}{r}\right)^2$$

$$F_c = 16{,}980psi - 0.53 \times \left(\frac{192in}{2.41in}\right)^2 = 13{,}616psi$$

This member will obviously qualify for use regarding compressive stress.

EXAMPLE NO. 6—JOIST AND STRINGER SPAN TABLES:

The following bridge deck falsework is to be checked using Span Tables for Joist and Stringers. There are two tables, the table **Table 5-393-200-9** on page 5-393.200(16), lists the maximum spans, in inches for structural members that are either Douglas Fir, No. 2 or Southern, Pine No. 2. The other table, **Table 5-393-200-10** on page 5-393.200(17), contains maximum span lengths, in inches, for Hem-Fir, No. 2, structural members. These tables show the maximum spans considering both strength and deflection. The following examples will illustrate the use of these tables.



Construction details given for Problem No. 6.

1. Stringers:

Determine applied loads per square foot:

Concrete:

$$8.25in \times \left({}^{1ft}\!\!\diagup\!_{12in} \right) \times 150lb / ft^3 = 103.1psf$$

Plywood:

$$0.75in \times \left({}^{1ft}\!\!\diagup\!_{12in} \right) \times 40lb / ft^3 = 2.5psf$$

2" x 6" Stringers:

$$2.3lb / ft \times \frac{1}{2ft(spacing)} = 1.2psf$$

Live load:

= 50.0 psf

Dead Load + Live Load, w = 156.8 psf

Determine the applied load in pounds per foot on each stringer:

Uniform Load = (stringer spacing) x w = 2 ft x 156.8 psf = 313.6 lb/ft

Using **Table 5-393-200-9** on page 5-393.200(16), the maximum span for a 2 x 6 Douglas Fir, No. 2 stringer is about 62 inches, which is greater than the 48 inch proposed spacing, so the member is acceptable for strength (bending and sear) and deflection. The stringer bearing stress should also be checked as indicated in the previous examples contained in this manual.

2. Joist:

Determine applied loads per square foot:

Stringer (dead load and live load):

= 156.8 psf

Joist (double 2 x 10):

$$2 \times 3.9 lb / ft \times \frac{1}{4\, ft(spacing)} = 2.0\, psf$$

Total: w, = 158.8 psf

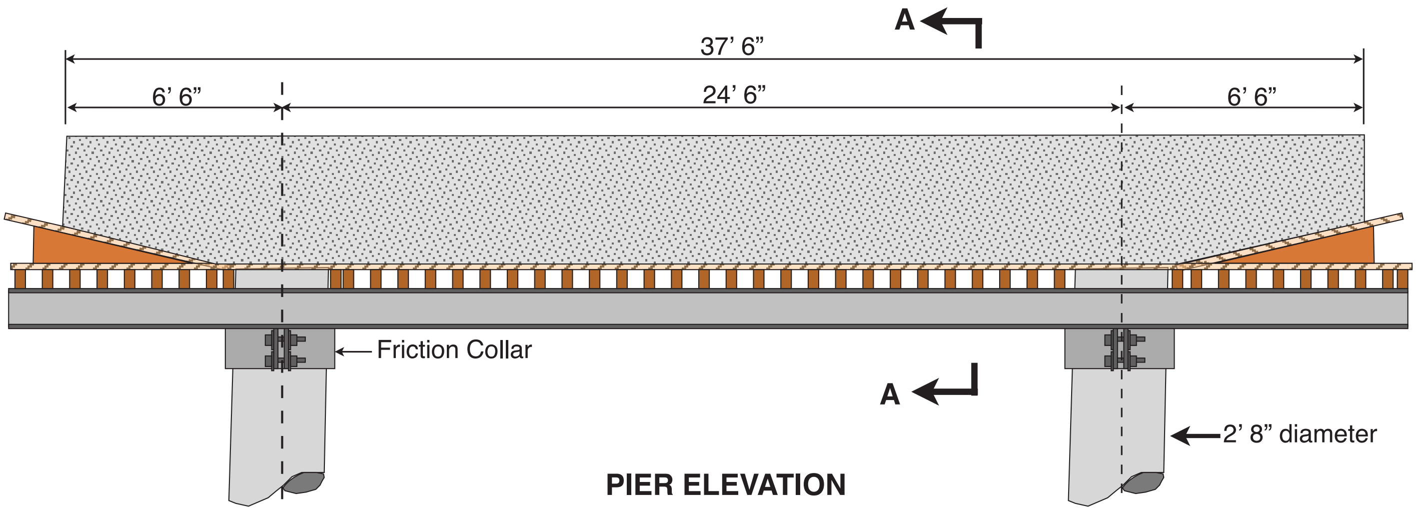
Determine the applied load in pounds per foot of joist:

Uniform Load = (joist spacing) x w = 4.0 ft x 158.8 psf = 635.2 lb/ft

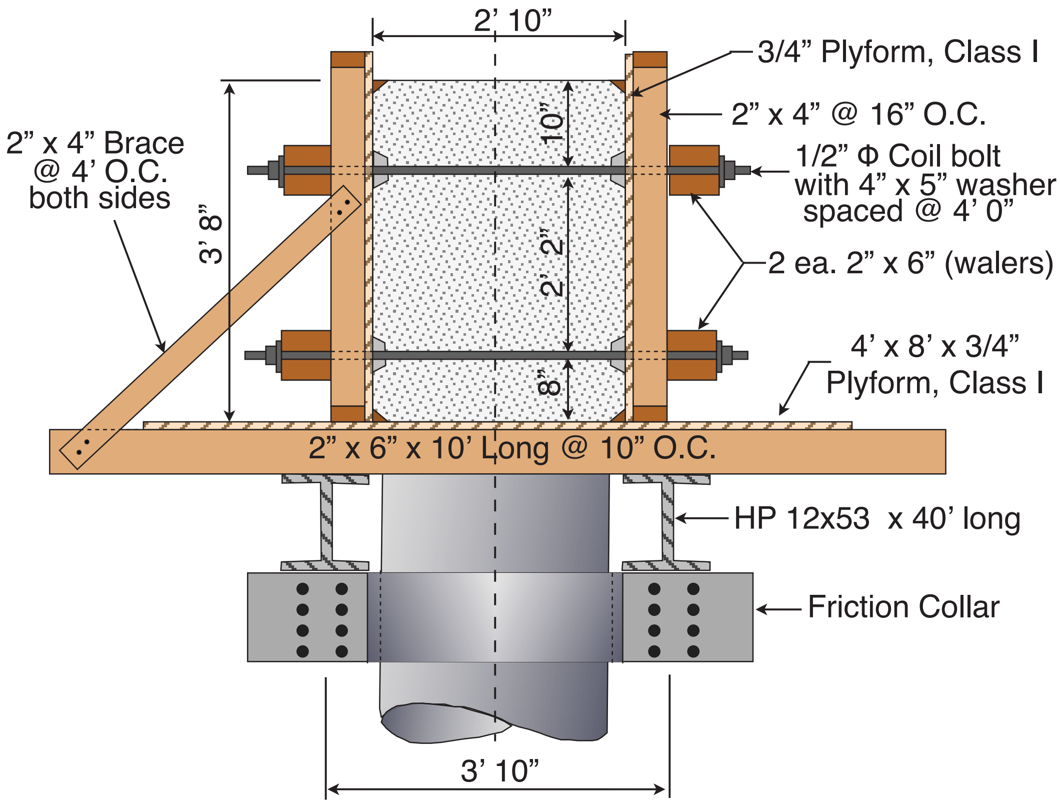
Again, using **Table 5-393-200-9** on page 5-393.200(16), the maximum span for the double 2 x 10 joist is determined by dividing the uniform load by two, as there are two members and the table is based on a single member. Reading the maximum spacing based on 318 lb/ft, one finds 96 inches. The effective span is 104 inches that is slightly greater than the allowable. The spacing of the joist should be reduced slightly in this example. Additionally, the bearing stress in the lumber and the stress in the hanger hardware should be checked as indicated elsewhere in this manual.

EXAMPLE NO. 7—PIER CAP FORM:

Assume the Contractor has proposed that the pier cap used in Example No. 1, seen on **page 5-393.200(41)**, will be used. Further assume, that all lumber used will be Douglas fir, No. 1. The members that will require stress investigation are as follows: (NOTE: Items defined as falsework were checked in Example No. 1.)



PIER ELEVATION



SECTION A-A

The following stress investigation would be necessary:

1. Sheathing
    a. Bending stress
    b. Rolling shear stress
    c. Deflection
2. Studs (2 x 4)
    a. Bending stress
    b. Horizontal shear stress
    c. Bearing on walers
    d. Deflection
3. Walers
    a. Bending stress
    b. Horizontal shear stress
    c. Bearing stress for tie plates
    d. Deflection
4. Tie Rods
    a. Tension stress or manufacturer safe load

Calculations follow:

The lateral concrete pressure is only load applied on the forms. About 17 cubic yards of concrete are required for the pier cap. Assume the Contractor anticipates placing this concrete in about a thirty minute period.

The rate of placement would be:

$$R = \frac{3.67\,ft}{30\,\min utes} = 7.34\,feet\,/\,hour$$

The formula for walls with a height less than 14 feet and with a rate of pour exceeding 7 feet/hour will apply.

where:

T = 70°F

C_C = 1.0

C_W = 1.0

R = 7.34 ft/hr

$$p_{\max} = C_C C_W \left[150 + \frac{43{,}400}{T} + \frac{2{,}800R}{T}\right] \leq wh$$

$$p_{\max} = \left[150 + \frac{43{,}400}{70} + \frac{2{,}800 \times 7.34\,ft\,/\,hr}{70}\right] = 1{,}064\,psf$$

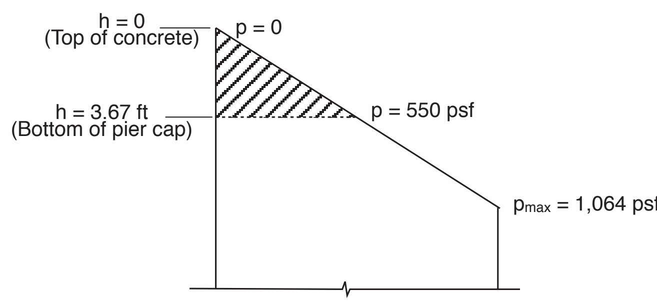
p_{max} = 1,064 psf (maximum pressure at any depth)

wh = 150 lb/ft^3 x 3.67 ft = 550.5 psf **(Controls)**

Use p = 550 psf

p = 150h = 150 x 3.67 ft = 550 psf (at bottom of cap form)

The following graph illustrates the relationship between height and pressure represented by the first equation as presented as a pressure diagram. Only the cross-hatched portion of the pressure diagram applies in this example.



Graphic representation of of the formula
p = 150h = 150 x 3.67 ft = 550 psf

1. Sheathing:

When a triangular shaped pressure diagram is involved, check the sheathing for the maximum pressure. In this case, check the sheathing for a pressure of 550 psf on a stud spacing of 16 inches. The sheathing material is 7/8 inch Plyform Class I. The chart for face grain across the supports (the strong direction), **Figure 5-393-200-15** on page 5-393.200(27), indicates that 7/8 inch Plyform with 16 inch stud spacing can safely carry just 550 psf. It must be verified later that the Contractor actually places the Plyform the "strong way."

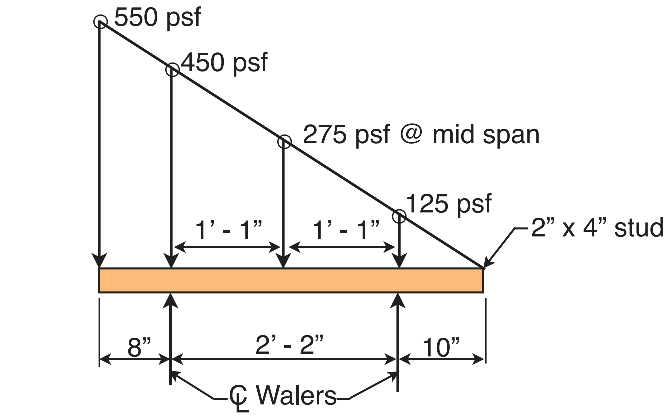
2. Studs (2 x 4):

The studs in this example should be checked as a simple span. In the following sketches, the studs will be shown horizontal to more clearly illustrate its beam action.

a. Bending Stress:

The pressure at mid span (275.0 psf) may be used as a uniform load for computing bending moments. The results will be slightly more conservative than would result from use of the actual loading.

Calculate uniform load on each stud:



Load and Pressure diagram for wall stud for pier cap form.

w = 275.0 psf x 16 in x (1 ft/12 in) = 366.7 lb/ft

Calculate bending moment in stud:

$$M = \frac{wl^2}{8} = \frac{366.7 lb/ft \times (2.167 ft)^2}{8} = 215.2 ftlb$$

Determine the Section Modulus of the 2 x 4 S4S stud using **Table 5-393-200-2**.

$S = 3.06$ in^3

$$f_b = \frac{M}{S} = \frac{215.2 ftlb \times \left(\frac{12in}{1ft}\right)}{3.06 in^3} = 843.9 psi \leq 1{,}375 psi$$

The actual bending stress is less than the allowable for Douglas Fir, No. 1, see **Table 5-393-200-3**.

b. Horizontal Shear Stress:

This should be checked by assuming the load at the left support (450 psf) extends uniformly across the simple span for calculating the maximum vertical shear force. Results will be slightly more conservative than would result from the use of the actual loading.

First, convert the pressure load to a uniform load for the studs spaced at 16 inches.

$$w = 450 psf \times 16 in \times \left(\frac{1ft}{12in}\right) = 600 lb/ft$$

Calculate the maximum vertical shear:

$$V = \frac{w(L - 2d)}{2}$$

$$V = \frac{600 lb/ft \times \left(2.167 ft - \left(2 \times 3.5 in \times \left(\frac{1ft}{12in}\right)\right)\right)}{2} = 475.1 lb$$

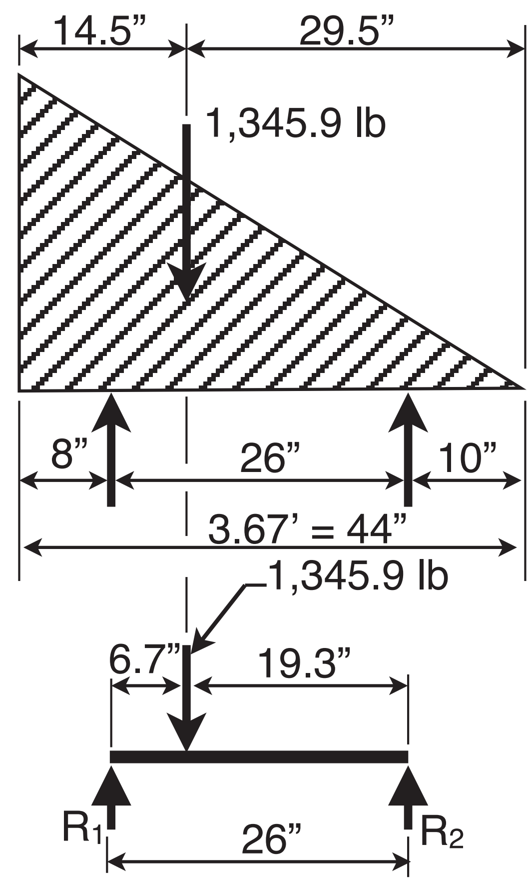
Calculate the horizontal shear stress.

$$f_v = \frac{3V}{2bd}$$

$$f_v = \frac{3 \times 475.1 lb}{2 \times 1.5 in \times 3.5 in} = 135.7 \leq 220 psi$$

The actual horizontal shear stress is less than the allowable for Douglas Fir, No. 1, see **Table 5-393-200-3**, the studs are adequate for horizontal shear.

c. Bearing Stress of Studs on Walers:



Pressure and load diagram for determining load on walers.

The maximum reaction will be at the lower waler. Actual reactions at each waler can be determined as follows:

Then total weight of the pressure block on each studs is:

$$\left(\frac{550lb/ft \times 3.67ft}{2}\right) \times 16in \times \left({}^{1ft}\!\!/_{12in}\right) = 1{,}345.7lb$$

The centroid of the pressure triangle is 1/3 of the distance from the base.

$$\frac{44.0in}{3} \approx 14.5in$$

Calculate the reaction from the pressure triangle on the lower waler.

1,345.7 lb x 19.3 in = R_1 x 26 in

$R_1 = P$ = 998.9 lb

R_2 = 1,345.7 lb – 998.9 lb = 346.8 lb

Determine the contact area using **Figure 5-393-200-17.**

Contact area, A = 4.5 in^2

Calculate bearing stress:

$$f_p = \frac{P}{A} = \frac{998.9lb}{4.5in^2} = 222.0psi \le 625psi \quad \text{OK!}$$

The bearing stress of the studs on the waler is less than the allowable side bearing stress of 625 psi.

d. Deflection of Stud:

Use the same loading condition used for determining the maximum bending stress.

$$\Delta = \frac{5wl^4}{384EI}$$

where:

w = 366.7 lb/ft

l = 2.167 ft

E = 1,700,000 psi

I = 5.36 in^4

$$\Delta = \frac{5wl^4}{384EI}$$

$$\Delta = \frac{366.7lb/ft \times (2.167ft)^4 \times \left({}^{12in}\!\!/_{1ft}\right)^2}{384 \times 1{,}700{,}000psi \times 5.36in^4} = 0.004in$$

Calculate allowable deflection:

$$\frac{2.167ft \times \left({}^{12in}\!\!/_{1ft}\right)}{270} = 0.096in \ge 0.004in \quad \text{OK!}$$

The studs are, therefore, acceptable with regard to deflection.

3. Walers:

The bottom waler will be checked since the higher stud reaction was found to exist at that location. A condition of uniform loading may be assumed to exist since three studs bear on each waler span (between tie rods).

The maximum reaction for a stud on the bottom waler was determined in Item 2 c above to be 998.9 lb. The stud spacing is 16 inches so the maximum reaction must be converted to a uniform load on the waler.

$$w = 998.9lb \times \left({}^{12in}\!\!/_{16in}\right) = 749.2lb/ft$$

a. Bending Stress in Waler:

The waler span length is equal to the tie rod spacing (4 feet). This member will be continuous over two or more spans. In keeping with the recommended simplifications, the assumption of simple spans may be used here.

Determine the bending moment:

$$M = \frac{wl^2}{8} = \frac{749.2lb/ft \times (4.0ft)^2}{8} = 1{,}498.4ftlb$$

Determine the Section Modulus using **Table 5-393-200-2**:

Section Modulus for two 2 x 6s : S = 2 x 7.56 in^3 = 15.12 in^3

Calculate bending stress using the following formula:

$$f_b = \frac{M}{S}$$

$$f_b = \frac{1{,}498.4\,ftlb \times \left(12in/1ft\right)}{15.12in^3} = 1{,}189.2\,psi \le 1{,}375\,psi$$

The actual bending stress is less than the allowable bending stress for Douglas Fir, No. 1, see **Table 5-393-200-3**.

b. Horizontal Stress in Waler:

First, calculate the maximum vertical shear force on the lower waler.

$$V = \frac{w(l-2d)}{2}$$

$$V = \frac{749.2lb/ft \times \left(4.0ft - \left(2 \times 5.5in \times \left(1ft/12in\right)\right)\right)}{2} = 1{,}155.0lb$$

Calculate the horizontal shear stress in the lower waler:

$$f_b = \frac{3V}{2bd}$$

$$f_b = \frac{3 \times 1{,}155.0lb}{2 \times 2 \times 1.5in \times 5.5in} = 105\,psi \le 220\,psi$$

This is less than the allowable horizontal shear stress for Douglas Fir, No. 1, see **Table 5-393-200-3**.

c. Bearing Stress in Waler from Plate Washer:

First, calculate the maximum reaction from lower waler on the plate washer.

P = 749.2 lb/ft x 4.0 ft = 2,997 lb

Using the center chart contained in **Figure 5-393-200-17** on page 5-393.200(38), determine the contact area for a 4 in by 5 in plate washer.

Contact area, A = 15.0 in^2

Calculate the bearing stress using the formula below:

$$f_v = \frac{P}{A} = \frac{2{,}997.8lb}{15.0in^2} = 199.8\,psi \le 625\,psi$$

This is less than the allowable compression perpendicular to grain stress, side bearing stress, for Douglas Fir, No.1. see **Table 5-393-200-3**.

d. Deflection of Waler:

Deflection will be calculated assuming the waler to be a simple span.

$$\Delta = \frac{5wl^4}{384EI}$$

where:

w = 749.2 lb/ft

l = 4.0 ft (spacing of tie rods)

E = 1,700,000 psi (Douglas Fir, No. 1, see **Table 5-393-200-3**)

I = 2 x 20.80 in^4 = 41.60 in^4 (**Table 5-393-200-2**)

$$\frac{5 \times 749.2lb/ft \times (4.0ft)^4 \times \left(12in/1ft\right)^3}{384 \times 1{,}700{,}000\,psi \times 41.60in^4} = 0.061in \le 0.125in$$

This surface is exposed to view. The allowable deflection of the span will be 1/8 inch since the L/270 value for this span is greater than 1/8 inch.

$$\frac{L}{270} = \frac{4.0ft \times \left(12in/1ft\right)}{270} = 0.178in$$

The actual deflection is less than the allowable deflection; therefore, the member is acceptable with regard to deflection.

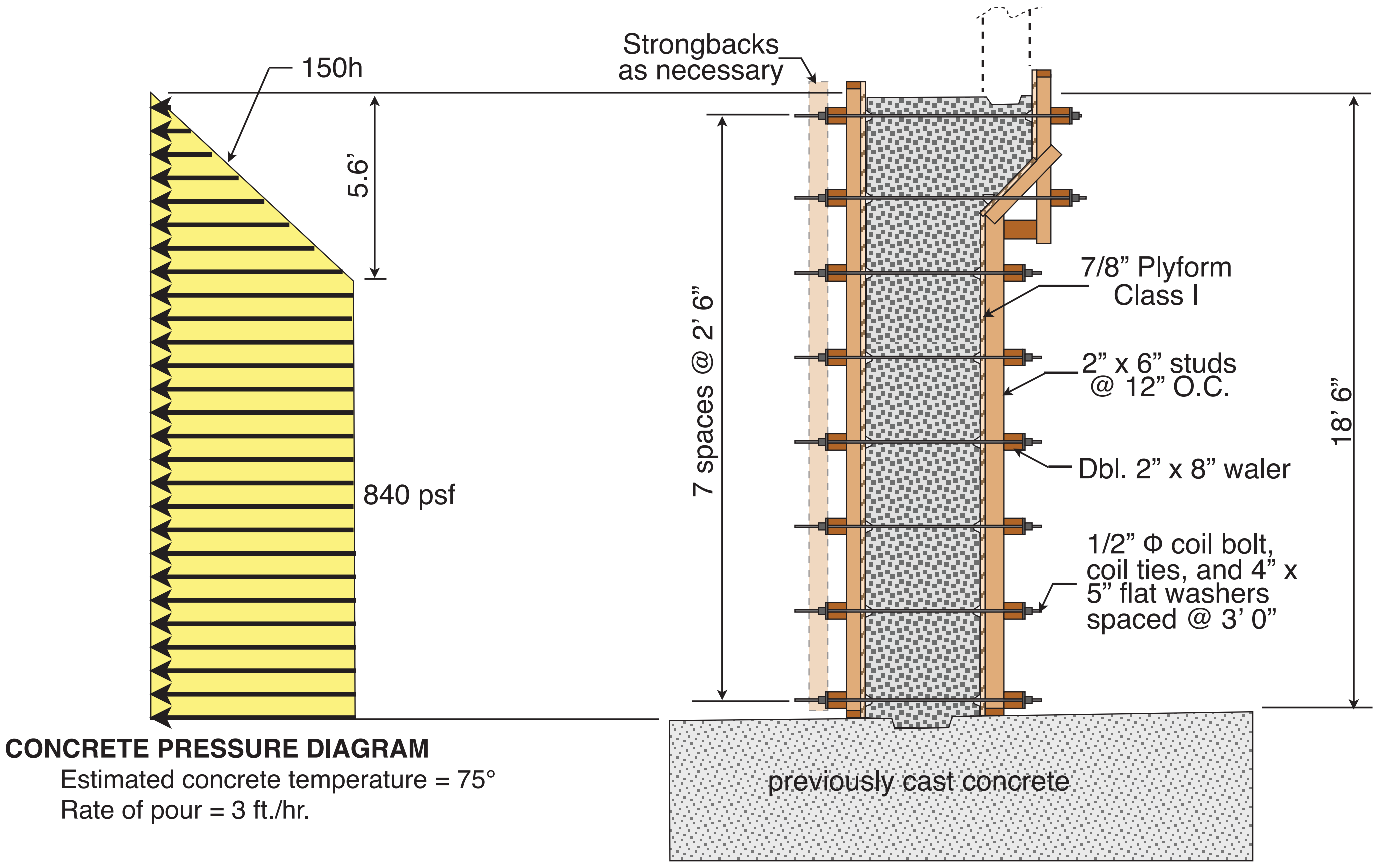
4. Tie Rods:

The maximum load on the tie rod was calculated as the reaction on the plate washer in Item 3 c above, P = 2,247.6 pounds.

The form plan indicates that ½ inch diameter coil bolts (and coil ties) will be used as form ties. The manufacturer's literature must indicate a load capacity of at least 2,997 pounds for the coil bolts and coil ties.

EXAMPLE NO. 8—ABUTMENT WALL FORM:

A check for the abutment forms as shown in the included diagram would require the following investigations. The anticipated rates of concrete placement are indicated on the figure.



CROSS SECTION OF ABUTMENT MAIN WALL FORMS

The following stress investigation would be necessary:

1. Sheathing
   a. Bending stress
   b. Rolling shear stress
   c. Deflection
2. Studs (2 x 6)
   a. Bending stress
   b. Horizontal shear stress
   c. Bearing on walers
   d. Deflection
3. Walers
   a. Bending stress
   b. Horizontal shear stress
   c. Bearing stress from tie plates
   d. Deflection
4. Tie Rods
   a. Tension stress or manufacturers safe load

The stress investigations listed above will be necessary for both the main wall and the parapet wall forms.

Calculations for the main wall forms are as follows:

First determine the amount of pressure from the fresh concrete on the forms. The Contractor has indicated a proposed rate of pour of 3 feet per hour in this example. Assume this concrete will be placed in mid-July, an anticipated temperature of 75° F may be used. Additionally, assume that the pressure coefficients C_W and C_C both can be considered equal to 1.0.

The formula for walls with a height greater than 14 feet and with a rate of pour less than 7 feet/hour will apply.

Assume: T = 75°F, C_C = 1.0, C_W = 1.0

R = 3 ft/hr

$$p_{\max} = C_C C_W \left[150 + \frac{43{,}400}{T} + \frac{2{,}800R}{T}\right] \le wh$$

$$p_{\max} = \left[150 + \frac{43{,}400}{75} + \frac{2{,}800 \times 3.0\,ft/hr}{75}\right] = 840\,psf$$

p_{max} = 840 psf (maximum pressure at any depth)

wh = 150 lb/ft^3 x 18.5 ft = 2,775 psf

design pressure, p = 840 psf

$$h = \frac{840\,psf}{150\,psf} = 5.6\,ft$$

1. Sheathing:

The pressure diagram indicates that the design uniform pressure is 840 psf. The sheathing material is 7/8 inch Plyform Class I. The chart for face grain across the supports (the strong direction), is found in the upper right corner of **Figure 5-393-200-15** on page 5-393.200 (37). That chart indicates that 7/8 inch Plyform Class I with 12 inch stud spacing can safely carry a little less than 800 psf. The imposed load of 840 psi is about 5% greater than chart value. This is close enough and the proposed sheathing can be used. It must be verified later that the Contractor actually places the Plyform the "strong way."

2. Studs:

The 2 x 6 studs are spaced at one foot with a uniform load of 840 psf. The span length is 2' – 6" (spacing of the walers). Assume the studs are continuous for more than three spans.

a. Bending Stress in Studs:

First, calculate the bending moment using the following formula:

M= 0.1wl^2 ref.: **Figure 5-393-200-16**

$$M = 0.1 \times 840\,lb/ft \times (2.5\,ft)^2 = 525\,ftlb$$

Determine the Section Modulus of the 2 x 6 studs:

S = 7.56 in^3 ref.: **Table 5-393-200-2**

Calculate the bending stress:

$$f_b = \frac{M}{S} = \frac{525\,ftlb \times \left(12in/1ft\right)}{7.56in^3} = 833.3psi \le 1{,}250\,psi$$

The actual bending stress is less than the allowable bending stress for Douglas Fir, No. 2, therefore the member is acceptable with regard to bending.

b. Horizontal Shear Stress in Studs:

First, calculate the maximum vertical shear force in the stud using the following formula:

$$V = 0.6w(l - 2d)$$

ref.: **Figure 5-393-200-16** on page 5-393.200(37)

$$V = 0.6 \times 840\,psf \times \left(2.5\,ft - \left(2 \times 5.5\,in \times \left(1ft/12in\right)\right)\right) = 800\,lb$$

Calculate horizontal shear stress:

$$f_v = \frac{3V}{2bd}$$

$$f_v = \frac{3 \times 800\,lb}{2 \times 1.5in \times 5.5in} = 145.5\,psi \le 220\,psi$$

The actual horizontal shear stress is less than the allowable horizontal shear stress for Douglas Fir, No. 2, therefore, the member is acceptable with regard to horizontal shear.

c. Bearing Stress of Stud on Waler:

First, calculate the reaction of the stud on the waler using the following formula:

P = 1.1wl = 1.1 x 840 lb/ft x (2.5 ft) = 2,310 lb

A = 1.5 in x 1.5 in x 2 = 4.5 in^2

Calculate the bearing stress:

$$f_p = \frac{P}{A} = \frac{2{,}310lb}{4.5in^2} = 513psi \le 625psi$$

This is less than the allowable side bearing stress for Douglas Fir, No. 2.

d. Deflection of Studs:

Calculate the deflection using the following formula:

$$\Delta = \frac{0.0069wl^4}{EI}$$

where:

w = 840 lb/ft

l = 2.5 ft

E = 1,600,000 psi

I = 20.80 in^4

$$\Delta = \frac{0.0069 \times 840lb/ft \times (2.5ft)^4 \times \left(12in/1ft\right)^3}{1{,}600{,}000psi \times 20.80in^4} = 0.012in$$

The allowable deflection is:

$$\frac{L}{270} = \frac{2.5ft \times \left(12in/1ft\right)}{270} = 0.111in \ge 0.012in \text{ OK!}$$

Since the actual deflection is less than the allowable, the studs are acceptable with regard to deflection. However, cumulative deflection of the sheathing plus stud plus the walers must not exceed 1/8 inch to meet alignment and stiffness criteria.

3. Walers:

Tie rods are spaced at 3' – 0". Assume walers will be continuous for three spans or more and use the three span continuous formulas. Since studs are spaced at 12 inches, there are at least 3 studs on each waler span and condition of uniform load may be assumed on the walers.

First, calculate the uniform load on the waler.

Uniform load, w = 840 lb/ft x 2.5 ft = 2,100 lb/ft of waler

a. Bending Stress in Waler:

Calculate bending stress in waler:

M = 0.01wl^2 = 0.01 x 2,100 lb/ft x (3.0 ft)^2 = 189.0 ft-lb

Calculate Bending Stress using the following formula:

$$f_b = \frac{M}{S}$$

where:

M = 189.0 ft-lb

l = 3.0 ft

S = 2 x 13.14 in3 = 26.28 in3

$$f_b = \frac{M}{S} = \frac{189.0ftlb \times \left(12in/1ft\right)}{26.28in^3} = 86.3psi \le 1{,}250psi$$

This is less than the allowable bending stress for Douglas Fir, No. 2, therefore, the walers are acceptable with regard to bending.

b. Horizontal Shear Stress in Waler:

First, calculate the maximum vertical shear force on the waler using the following formula:

$V = 0.6w(l\text{-}2d)$

V = 0.6 x 2,100 lb/ft x (3.0 ft –(2 x 7.25 in) x (1 ft/ 12 in)) = 2,257.5 lb

Next, calculate horizontal shear stress using the following formula:

$$f_v = \frac{3V}{2bd} = \frac{3 \times 2{,}257.5lb}{2 \times 2 \times 1.5in \times 7.25in} = 155.7psi \le 220psi$$

The actual horizontal shear stress is less than the allowable horizontal shear stress for Douglas Fir, No. 2, therefore, the member is adequate with respect to horizontal shear.

c. Bearing on Tie Plate:

First, calculate the maximum reaction on the tie rod plate:

$P = R_2 = 1.1wl$ ref.: **Figure 5-393-200-16**

P = 1.1 x 2,100 lb/ft x 3.0 ft = 6,930 lb

Next, determine the contact area of the washer on the waler using the center chart in **Figure 5-393-200-17**. The contact area for a 4" x 5" plate washer with a ¾ inch space between the waler members, A = 9.0 in^2.

Calculation of bearing stress:

$$f_v = \frac{P}{A} = \frac{6{,}930lb}{13.75in^2} = 504\,psi \leq 625\,psi$$

This is less than the allowable side bearing stress for Douglas Fir, No. 2, therefore, the member is acceptable with regard to side bearing.

d. Deflection of Waler:

Calculation the deflection of the waler using the following formula:

$$\Delta = \frac{0.0069wl^4}{EI}$$

where:

w = 2,100 lb/ft

l = 3.0 ft

E = 1,600,000 psi

I = 2 20.80 in^4 = 41.60 in^4

$$\Delta = \frac{0.0069 \times 2{,}100lb \times (3.0\,ft)^4 \times \left(\frac{12in}{1ft}\right)^3}{1{,}600{,}000\,psi \times 41.60in^4} = 0.03in$$

The allowable deflection is:

$$\frac{L}{270} = \frac{3.0\,ft \times \left(\frac{12in}{1ft}\right)}{270} = 0.133 \geq 0.030in$$

Since the actual deflection is less than the allowable, 0.133 in, the walers are acceptable with regard to deflection. However, cumulative deflection of the sheathing plus stud plus waler must not exceed 1/8 inch (0.125 inch) to meet alignment and stiffness criteria.

Deflection of sheathing is negligible

Deflection of stud = 0.012 in

Deflection of waler = 0.030 in

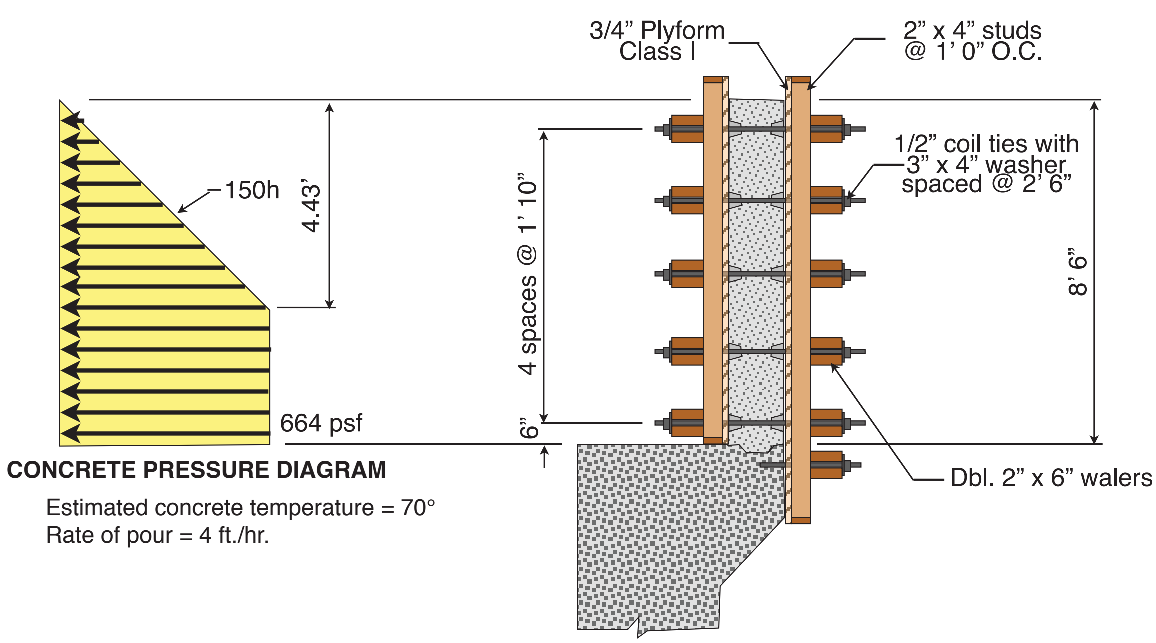
Cumulative deflection = 0.042 in < 0.125 in OK!

4. Tie Rods—Tensile Stress:

Tie load calculated for the reaction on the plate washer, 6,930 pounds, in Item 3c above will be used for the tensile load on the tie rod. The form details indicates that ½ inch diameter coil bolt and coil tie will be used. The manufacturer's literature must be checked to determine that these bolts and ties will carry the 6,930 pound load.

Parapet Wall Forms:

Calculations for checking the parapet wall forms shown in the diagram are as follows:



CROSS SECTION OF PARAPET WALL FORMS

First determine the amount of pressure from the fresh concrete on the forms. The form plan indicates a proposed rate of concrete placement of 4 feet per hour. Assuming concrete placement will be in late August, a concrete temperature of 70°F may reasonably be used. The formula for placement of concrete in wall less than 14 feet in height with a rate of fill less than 7 feet per hour will be used.

$$p_{\max} = C_W C_C \left[150 + \left(\frac{9{,}000R}{T}\right)\right] \le wh$$

where:

$C_W = 1.0$

$C_C = 1.0$

$R = 4$

$T = 70°\text{F}$

$h = 8.5$ ft

$$p_{\max} = \left[150 + \left(\frac{9{,}000 \times 4\,ft}{70}\right)\right] = 664$$

wh = 150 psf x 8.5 ft = 1,275 psf > 664 psf

p = 664 psf (maximum pressure at any depth)

The design pressure, p = 664 psf

Solve for h: h = 664 psf/150 psf = 4.43 feet

The resulting concrete pressure diagram is shown in the details shown for this example. The actual stress calculations for the parapet wall forms will be similar to those for the main wall forms and, therefore, will not be repeated in this example. However, it would be necessary to perform these calculations sine the concrete pressure and member spacing differ from those of the main wall forms.

5-393.209 Glossary

ANCHOR—form anchors are devices used to secure formwork to previously placed concrete of adequate strength, normally embedded in concrete during placement

BASE PLATE—a device used between post, leg, or screw jack and foundation to distribute the axial load

BATTEN (BATTEN STRIP)—a narrow strip of wood placed on the back side of a vertical joint of sheathing or paneling, or used on the inside form surface used to produce architectural affects in the concrete; also called a *cleat*

BATTER—inclination from vertical, generally stated as offset in inches per foot vertically

BEAM HANGER—a wire, strap, or other hardware device that supports formwork from structural members such as bridge beams

BENT—two-dimensional frame which is self-supporting within these dimensions. It has at least two legs and is usually placed at right angles to the structure which it carries.

BLOCK—a solid piece of wood or other material used to fill spaces between formwork members

BUGHOLES—small regular or irregular voids in the formed concrete surface, resulting from air trapped during placement and consolidation of concrete; also called *blowholes*.

BULKHEAD—a partition in the forms blocking fresh concrete from a section of the forms or closing the end of the form, such as a construction joint.

CAMBER—a slight (usually) upward curvature of a truss, beam, or form to improve appearance or to compensate for anticipated dead load deflection.

CATWALK—a narrow elevated walkway

CENTER MATCHED—tongue-and-groove (T&G) lumber with the tongue and groove at the center of the piece rather than offset as in standard matched

CHAMFER—refers to a beveled corner which is formed in the concrete work by placing a three-cornered piece of wood in the form corner

CHAMFER STRIP—triangular or curved insert placed in inside corner of form to produce rounded or beveled corner; also called *fillet* or *cantstrip*

CLEANOUT—an opening in a form for removal of refuse and is closed before the concrete is placed

COATING—material applied by brushing, dipping, mopping, or spraying, etc. to preserve form material and to facilitate stripping

COLUMN CLAMP—any of various types of tying or fastening units used to hold column form sides together

CONTRACTION JOINT—formed, saw-cut or tooled groove in concrete structure to regulate location of shrinkage cracks

CONSTRUCTION JOINT—the surface where two successive placements of concrete meet, frequently with a keyway or reinforcement across the joint

CRUSH PLATE—an expendable strip of wood attached to the edge of a form or intersection of fitted forms, to protect the form from damage during prying, pulling, or other stripping operations, the term is also used to designate a *wrecking strip*.

DECK—the form upon which concrete for a slab is placed

DECKING—sheathing material for a deck or slab form

DIAGONAL BRACING—supplementary (not horizontal or vertical) formwork members designed to resist lateral load

DRESSED LUMBER—meaning lumber that has been surfaced (planed) to standard dressed sizes. If surfaced on all four sides is denoted as S4S, if surfaced on two sides is denoted as S2S

DRIP—a cutout in the under side of a projecting piece of concrete to prevent water from working its way back to the supporting element

DRY TIE—form tie that holds sides of forms in position in an area where no concrete is placed, for example at the top of a wall form above a construction joint

FALSEWORK—the temporary structure to support work in the process of construction, in concrete construction is considered to be the framework required to maintain a concrete unit in the desired position (when it cannot be supported on the ground, as a footing or on previously cast concrete) until the concrete is strong enough to carry its own dead weight

FASCIA—a flat member or band at the exposed face of the deck of a bridge or the edge beam on a bridge

FILLER—material used to fill an opening in forms

FILLET—*see chamfer strip*

FIN—narrow linear projection from formed concrete surface, resulting from mortar flowing into or through horizontal or vertical joints in the formwork

FORMS—those members (usually vertical) that required to maintain plastic concrete in its desired shape until it has set up, forms resist the fluid pressure of the plastic concrete, the additional fluid pressure generated by mechanical vibration of the concrete and the impact of placing the concrete

FORM FINISH—finish of a concrete surface that has not been altered since removal of the forms, also referred to as an *as-cast finish*

FORMWORK—the total system of support for freshly placed concrete including the mold or sheathing as well as all supporting members, hardware, and necessary bracing

FORMWORK ENGINEER—responsible profession engineer in charge of the design and construction of the formwork

FULL SAWN LUMBER—meaning lumber that was sawn green to sizes greater than the standard sawn sizes so that in use the lumber will be same as the nominal size.

GLUED LAMINATED TIMBER (GLULAM)—an assembly of selected suitably prepared lumber laminations bounded together with adhesives, with the grain of the laminations approximately parallel longitudinally, that are fabricated in compliance with ANSI/AITC A190.1

HANGER—a device used for suspending one object from another, such as hardware used to support joists from the upper flange of bridge beams

HAUNCH—that portion girder, arch, or deck that is thickened near the support, also a bracket on a wall or column, used to support a load outside the wall or column, also referred to as a corbel

HONEYCOMB—irregular voids left at a formed concrete surface where the mortar fails to effectively fill spaces between coarse aggregate particles

HORIZONTAL BRACING—horizontal load-carrying members attached to formwork components to increase lateral load resistance; when attached to shores they may also reduce the shores' unsupported length, thereby increasing load capacity and stability

INVERT—lowest visible surface; the floor of a drain, sewer, or gutter line at front face of curbs

JACK—mechanical device used for adjusting elevation of forms or form supports

JOIST—a horizontal member supporting deck form sheathing, usually rests on stringers or ledgers

KEYED—fastened or fixed in position in a notch or other recess

KEYWAY—a recess or groove in one lift or placement of concrete which is filled with concrete of the next lift, giving shear strength to the joint; also called a key

KICKER—a piece of wood (block or board) or metal attached to formwork member to take the trust of another member; sometime called a *cleat*

KNEE BRACE— brace between horizontal and vertical members in a formwork to make the formwork more stable; in formwork it acts as a haunch

LACING—horizontal brace between shoring members

LAGGING—designates heavier timber sheathing used between soldier piles; used in retaining walls and underground work to resist soil pressure; see *soldier piles and lagging*

LAMINATED VENEER LUMBER (LVL)—a structural composite lumber product manufactured from veneers that typically is about 1/8 in. thick, laminated so that the grain of all veneers runs parallel to the axis of the member; bonded with an exterior adhesive

LEDGER— horizontal formwork members, especially one attached to a beam side that supports the joist; also may be called girt, sill, purlin, stringer

LINING—any sheet, plate, or layer of material attached directly to the inside face of the forms to improve or alter the surface texture and quality of the finished concrete; also called liner

NOMINAL LUMBER DIMENSIONS—the cross-section dimensions of lumber in inches as a full sawn piece (dimension prior to surfacing).

MOLD—the cavity or surface against which fresh concrete is cast to give it a desired shape; sometimes used interchangeably with *form*

MUDSILL—a plank, frame, or small footing on the ground used as a base for a shore or post in formwork

NAILER—strip of wood or other fitting attached to or set in concrete, or attached to steel, to facilitate making nailed connections

NEAT LINE—a line defining the proposed or specified limits of an excavation or structure

NOSING—a projection, such as the projection of the tread of a stair over the riser

OFFSET—an abrupt change in alignment or dimension, either horizontally or vertically

ORIENTED STRAND BOARD (OSB)—a panel product made of layers of thin wood strands bonded with waterproof resin under heat and pressure. Strands of each layer are aligned parallel to one another, but perpendicular to those in adjacent layers.

OVERBREAK—excavation beyond the neat line of a tunnel or other structure

PANEL—a section of form sheathing, constructed from boards plywood, metal sheets, etc., that can be erected and stripped as a unit

PARAPET—the part of the wall that extends above the roof level, or above the bridge seat

PARALLEL STRAND LUMBER (PSL)—a structural composite lumber wood product made by gluing together, parallel to the products length, long strands of wood that have been cut from softwood veneer

PERMANENT FORM—any form that remains in place after the concrete has developed its design strength. The form may or may not become an integral part of the structure. Also may be called a *stay-in-place form*.

PILASTER—column built within a wall usually projecting from the wall

PLATE—flat horizontal member at the top and/or bottom of studs or posts

PLUMB—vertical, or the act making vertical

POST—vertical formwork member used as a brace, also shore, prop, jack

PLYFORM—is an exterior-type plywood designed to be used as form material

PLYWOOD —is a panel wood product consisting of odd numbers of plies, each placed at right angles to the adjacent plies and bonded together with glue

REVAL—the side of an opening in a wall

RIBS—parallel structural members backing sheathing

ROUGH SAWN LUMBER—lumber that has been sawn to sizes that are approximately 1/8 inch larger that the standard lumber sizes

RUSTICATION—a groove or series of grooves in a concrete surface

RUSTICATION STRIP—a strip of wood or other material attached to a form surface to produce a groove or rustication in the concrete

SCAB—a small piece of wood fastened to two formwork members to secure a butt joint

SCAFFOLD or SCAFFOLDING—a temporary elevated platform (```````````````` ``supported or suspended) and its supporting structure used to support workers, tool, and materials

SCREED—tow or more strips set at the desired elevation so that the concrete may be leveled by drawing a straightedge over their surface

SCREEDING—the operation of pulling a straightedge over the surface of the screeds thus leveling the concrete

SCREWJACK—a load-carrying device composed of a threaded screw and an adjusting handle used for vertical adjustment of shoring and formwork

SHEATHING—the supporting layer of formwork closest to the concrete; either in direct contact with the concrete or separated from it by a liner

SHORE—a temporary vertical or inclined support for formwork and fresh concrete. Also called prop, post, strut

SHORING—system of vertical or inclined supports for forms; may be wood or metal posts, scaffold-type frame, or various patented members

SLIPFORM—also referred to as *sliding form*. A form which moves, usually continuously, during placing of the concrete

SNAP TIE—concrete wall form tie, the end of which can be twisted or snapped off after the forms have been removed

SOFFIT—the underside of a subordinate part or member, such as the deck overhang

SOLDIER PILES AND LAGGING—steel H piles driven to the required depth before excavation. As excavation proceeds horizontal timber lagging is placed against the face of the excavation and wedged between the flanges of the H piles.

SOLDIERS—vertical wales used for strengthening or alignment

STIFFBACK—see strongback

STRINGER—horizontal structural member usually (in slab forming) supporting joists and resting on vertical supports

STRONGBACK—a frame or member attached to the back of a form to stiffen or reinforce it; additional vertical wales placed outside horizontal wales for added strength or better alignment; are also called stiffback

STRUCTURAL COMPOSITE LUMBER—a generic term that describes a family of engineered wood products in which veneer sheets, strands, or other small wood elements are bonded together with exterior structural adhesives to form lumber-like materials

STUD—a member of appropriate size and spacing to support the sheathing of concrete forms; commonly used vertical

SWAY BRACE—a diagonal brace used to resist wind or other lateral force

TELLTALE—any device designed to indicate movement of formwork

TIE—a concrete form tie is a tensile unit adapted to holding concrete forms secure against lateral pressure of unhardened concrete, with or without provisions for spacing the forms a definite distance apart, and with or without provision for removal of metal to a specified distance back from the finished concrete surface

TIE HOLE—void in a concrete surface left when a tie end is snapped off, broken off or otherwise removed

TEMPLATE—thin plate or board frame used as a guide in positioning or spacing form parts, reinforcement, anchors, etc.

TOENAIL—to drive a nail at an angle thru the end of a wood member into another wood member, used to connect studs to the plate

TOLERANCE—the permitted variation from a given dimension, quantity, location, or alignment

WALE—a long horizontal formwork member (commonly double) used to gather loads from several (vertical) studs or similar members to allow wider spacing of restraining ties; when used with prefabricated panel forms. This member is used to maintain alignment; also called waler or ranger

WRECKING STRIP—small piece or panel fitted into formwork assembly in such a way that it can be easily removed ahead of main panels or forms, making it easier to strip these major form components

X-BRACE—paired set of (tension) sway braces

5-393.2010 Symbols and Units of Measurements

The following symbols, abbreviations and units of measurements will apply to Chapter 200—Forms and Falsework.

A = area, in2

a = dimension in beam diagrams, ft, in

b = width of beam, in

c = distance from neutral axis, in

C_C = Chemistry Coefficient used pressure calculations

C_D = Duration of load factor

C_r = Repetitive member factor

C_W = Unit Weight Coefficient used in pressure calculations

d = depth of beam, in

d = penny weight used to measure nails

e = total deformation, in

E = modulus of elasticity, psi

f = unit stress in member, psi

f_b = actual bending stress, psi

f_c = actual compressive stress, psi

f_p = actual bearing stress, psi

f_v = actual shear stress, psi

f_{rv} = actual rolling shear stress, psi

F'_b = allowable bending stress, psi

F'_c = allowable compressive stress, psi

F'_v = allowable shear stress, psi

h = depth of concrete, ft

I = moment of inertia, in4

K = column end coefficient

L = span length, in, ft

l = span length, in, ft

M = bending moment, ft-lb, in-lb

P = concentrated load, lb

R = rate of concrete pour, hr

R_1 = reaction at beam support, lb

S = span length, in, ft

s = unit deformation, in/in

T = temperature,°F

t = thickness, in

t_w = web thickness, in

w = unit weight, lb/ft

Δ = calculated deflection, in

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5. Minnesota Department of Transportation, Bridge Office, *LRFD Bridge Design Manual 5-392*, Oakdale, MN, September 2008