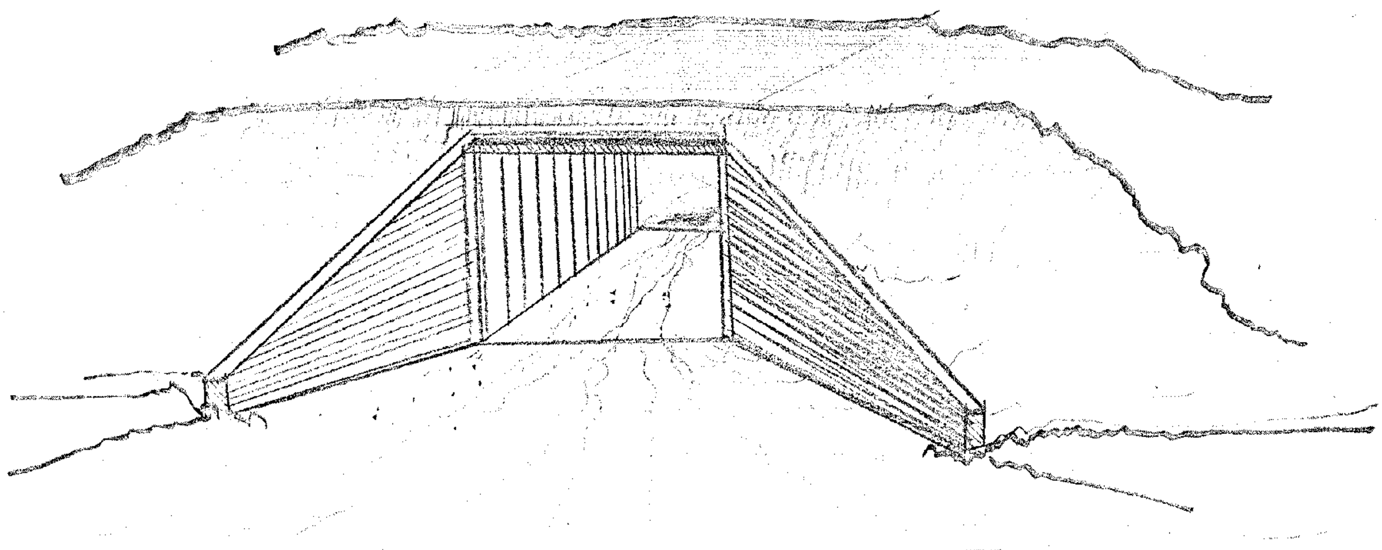
PB95-170262

Timber Substructures for Bridge Applications

Publication No. FHWA-RD-93-024

March 1994



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FOREWORD

This study is part of a research effort on low-cost timber bridge technology conducted by the Constructed Facilities Center at West Virginia University in cooperation with the Federal Highway Administration.

This publication discusses several timber substructural systems used in modern timber bridges. It offers specific design guidelines on timber piles, steel bent and steel-driven pile abutments, timber culverts, and timber crib abutments. This booklet is written primarily for design engineers and inspectors concerned with timber substructures. Two other similar booklets dealing with the behavior of stress-laminated timber decks and the corrosion protection of steel components used in modern timber bridges are also being produced under this research program.

Charles J. Nemmers, P.E.
Director, Office of Engineering and Highway
Operations Research and Development

NOTICE

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	ml
gal	gallons	3.785	liters	l
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 l shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	l
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
psi	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	ac
ha	hectares	2.47	acres	mi^2
km^2	square kilometers	0.386	square miles	
		VOLUME		
ml	milliliters	0.034	fluid ounces	fl oz
l	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg	megagrams	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	psi

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised August 1992)

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AASHTO .... American Association of State Highway and Transportation Officials
ASTM .... American Society for Testing and Materials
AISC .... American Institute of Steel Construction
AWPA .... American Wood Preservers' Association
FHWA .... Federal Highway Administration
NDS .... National Design Specification

ACKNOWLEDGEMENT

We would like to thank the following people for their contributions to this booklet: Hota GangaRao, Barry Dickson, and Penmatsa Raju (CFC-WVU) for their expert suggestions and contributions; Sheila Duwadi and the Timber Bridge Research Council (FHWA) for their valuable review and comments; the staff of the Constructed Facilities Center for their administrative support; and Alex Kharter (WVU) for sketching the cover illustrations.

CHAPTER 1. INTRODUCTION

PURPOSE AND SCOPE

A substructure supports the bridge superstructure and transfers the dead and live loads to the supporting soil or rock. The type of substructure used depends on the site conditions and the expected highway loads. Modern timber bridges are adaptable to any type of substructure constructed from concrete, steel, or timber. Discussions in this booklet are limited to substructural systems constructed from timber and steel-bent abutments with timber lagging.

Within the last decade, timber bridges have become a viable alternative for new bridge construction on low-volume roads, where it is imperative that the bridges be economical, long-lasting, and nearly maintenance free. Extensive research conducted at various universities and companies in the U.S. and Canada has provided practical recommendations and guidelines for innovative bridge superstructural systems.[1,2,3] At present, however, practical recommendations concerning timber substructural systems are not readily available. Specifically, information on design procedures, construction guidelines, and maintenance and rehabilitation are not well documented.

In response to this need, the objectives of this booklet are: (1) to present background information on timber substructural systems, (2) to present practical design guidelines for a selection of systems, (3) to recommend guidelines for construction and inspection procedures, and (4) to present sources of additional information on timber substructures for bridge applications.

Based on a survey of the most popular timber substructures used in bridge applications, the following five systems were selected: timber piles, bent-pile abutments, culverts, crib-wall abutments, and stub abutments. For some of these systems, specific designs were selected in cooperation with fabricators and highway departments. After contacting practicing engineers and professionals with expertise in timber substructures, it was concluded that design details for some of these systems were not readily available. Therefore, some of the designs presented in this manual have been developed entirely by the authors.

Information on timber piles is presented in chapter 2, which includes design procedures followed by a design example for a specific site, and a stub abutment overview. The designs of steel bent-pile abutments are given in chapter 3. Timber culverts are discussed in chapter 4, which also includes a detailed design procedure followed by a design example. Timber crib abutments are presented in chapter 5. Finally, inspection and quality control guidelines are presented in chapter 6. Each chapter contains sources of practical information that can be used by highway departments, designers, and inspectors.

CHAPTER 2. PILE FOUNDATIONS

OVERVIEW

If the soil beneath the level at which an abutment would normally be used is too weak or too compressible to provide adequate support, the loads are transferred to a more suitable stratum at a greater depth by means of piles. Bent-pile abutments are constructed of concrete, steel, or timber piles driven to a sufficient depth to develop the required load capacity by end-bearing, or through friction between the pile surface and surrounding soil. Back walls and wing walls are typically provided to retain the embankment fill material (figure 1). The bridge superstructure is connected to the piles by a continuous pile cap attached to the piles. The design methodology presented in this chapter is limited to abutments constructed of timber piles.

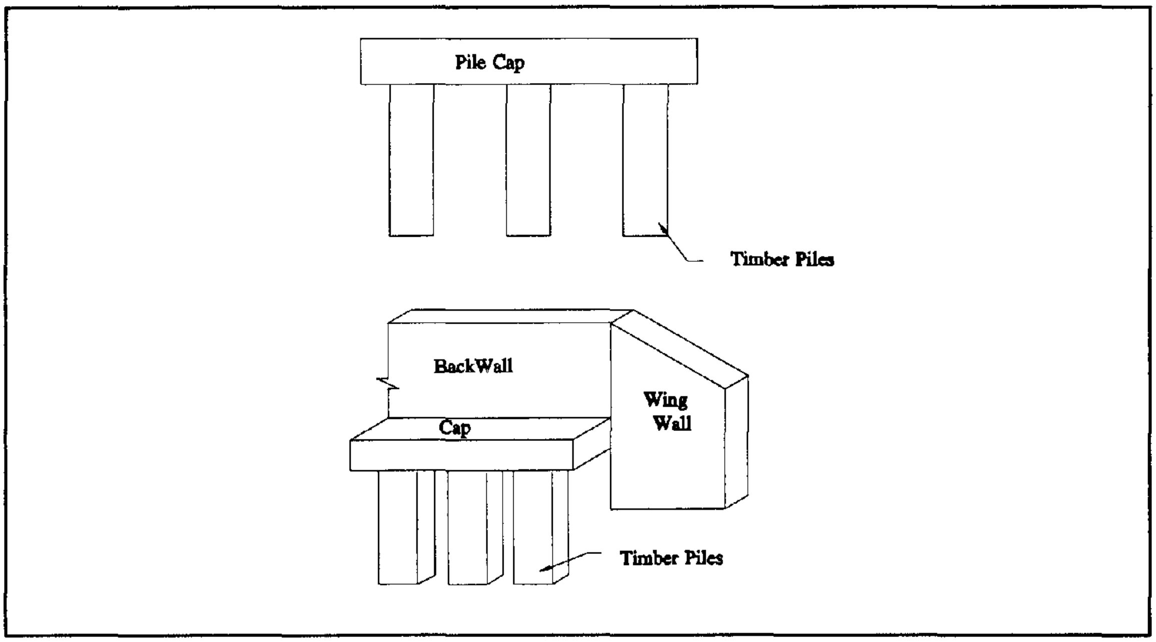

Figure 1. Bent pile abutment.

MATERIALS

Basically, timber piles consist of tree trunks treated with preservatives and driven into the soil using the small end as a point. The natural taper of the tree, which is about 1 in per 10 ft (25 mm per 3.05 m) for southern pine and 3/4 in per 10 ft (19.05 mm per 3.05 m) for douglas fir, facilitates the ground penetration process and compacts the soil around the pile, enhancing the pile's load-carrying capacity.

Round timber piles are supplied in accordance with ASTM D25-91 (AASHTO M168), which list the diameter and length requirements for friction piles (see ASTM D25-91 table 1, p. 2) and end-bearing piles (see ASTM D25-91 table 2, p. 4). The ASTM standard also specifies the size of allowable knots, shakes, and splits.[4] A typical timber pile is shown in figure 2. Round timber piles are specified either by the butt circumference measured at 3 ft (0.915 m) from the butt and a minimum tip circumference or by the tip circumference with a minimum butt circumference. Timber piles under ASTM D25-91 are designed at a critical section using allowable design stresses outlined in ASTM D2899. The critical section for a friction pile is located away from the tip, while an end-bearing pile may have its critical section at the tip.[5] For additional information on locating the critical section and critical stresses in round-tapered timber piles, see references 6 and 7.

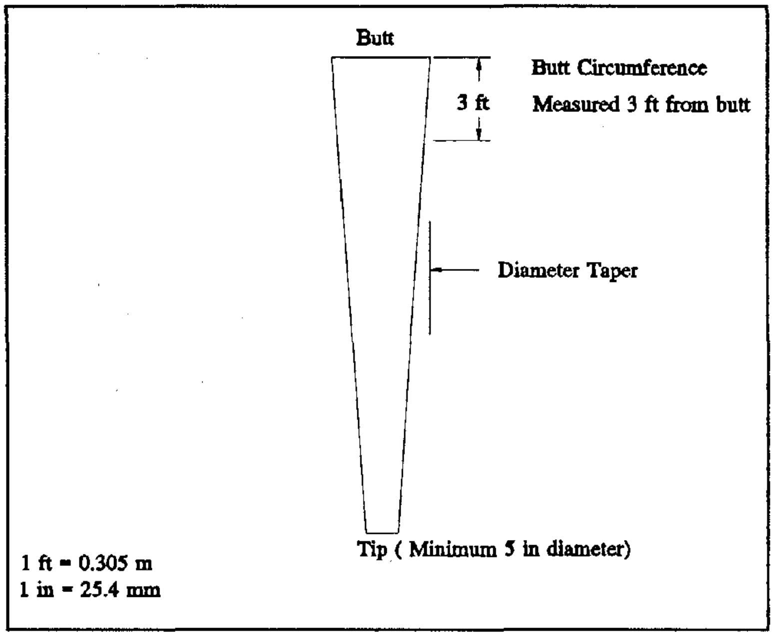

Figure 2. Typical timber pile.

Generally, southern yellow pine and douglas fir are the two species most often specified, which are available in lengths of up to 80 ft (24.4 m) and 125 ft (38.1 m), respectively. Oak is available in lengths of up to 60 ft (18.3 m). The allowable design stresses for douglas fir and southern pine are given in ASTM D-2899, which specifies a maximum compressive stress of 1,200 lbf/in^2 (8.26 MPa) for both species.

Timber caps should not be less than 10 in by 10 in (254 mm by 254 mm) and should be fastened to the piles with drift bolts not less than 3/4 in (14.28 mm) diameter, extending at least 9 in (228.6 mm) into the piles.

PRESERVATIVE TREATMENT

Untreated timber piles embedded at a level below the ground water table are considered permanent because decay-producing fungi cannot prosper without oxygen. However, when timber piles are projecting above the ground water table, the timber piles are subject to decay by fungi and attack by insects and marine borers, and therefore, piles should be treated with a preservative.

Timber piles are pressure-treated in accordance with AWPA standard C3-91 using creosote, water-borne salts, and pentachlorophenol. However, pentachlorophenol is only specified for fresh water use, since it is soluble in salt water. Information on treatment of southern pine, douglas fir, and other species is provided by AWPA, and as an example, for fresh water use, southern pine piles should be pressure-treated with coal tar creosote conforming to AWPA standards P1/P13-89 or P2-89. AWPA standard C3-91 specifies a retention level of 12 lbf/ft^3 (1.88 N/m^3) and a minimum penetration level of 3 in (76.2 mm) for southern yellow pine piles.[8]

AWPA standard C3-91 outlines specific requirements for steaming operations. For southern pine piles, steaming is required to ensure adequate creosote retention and penetration. For douglas fir, steaming is not permitted because it can damage the pile, and instead, Boultinizing (boiling in oil under a vacuum) is recommended.

Treated piles should be handled with care. Any cutting, framing, and drilling should be done before preservative treatment. All exposed surfaces due to handling and machining should be field-treated in accordance with AWPA standard M4-90.[9]

INSTALLATION

Piles are commonly driven by means of a hammer or by a vibratory force generator. Timber piles can be driven by any kind of pile hammer, provided the hammer meets the allowable size requirement. A heavy pile should be driven by a heavy hammer delivering large energy. For more information on selection of pile hammers, see references 10 and 11.

One of the drawbacks of timber piles is the possibility of damage due to overdriving. Timber piles may be damaged at the tip or butt. Therefore, the behavior of the pile and the blows per inch (25.4 mm) of penetration during pile driving should be monitored. Typically, to avoid pile damage during driving, a resistance of 4 to 5 blows per inch (25.4 mm) is recommended.[10] Furthermore, to minimize splitting and crushing of fibers at the top of piles, AWPA suggests strapping the timber pile at the top by steel straps or bands.[6] The protection of pile tips can be accomplished by the use of a steel-pointed shoe or a blunt pile tip. When using a steel-pointed shoe, the tip of the pile should be accurately shaped, otherwise the pile will be driven eccentrically.[6]

DESIGN CRITERIA

To establish a need for a pile foundation, the bridge designer must determine whether or not the site conditions are such that piles must be used. Some of the situations where pile foundations are used are shown in figure 3. The most common situation is that in which the upper soil strata is weak and too compressible to support heavy vertical loads transmitted by the superstructure.

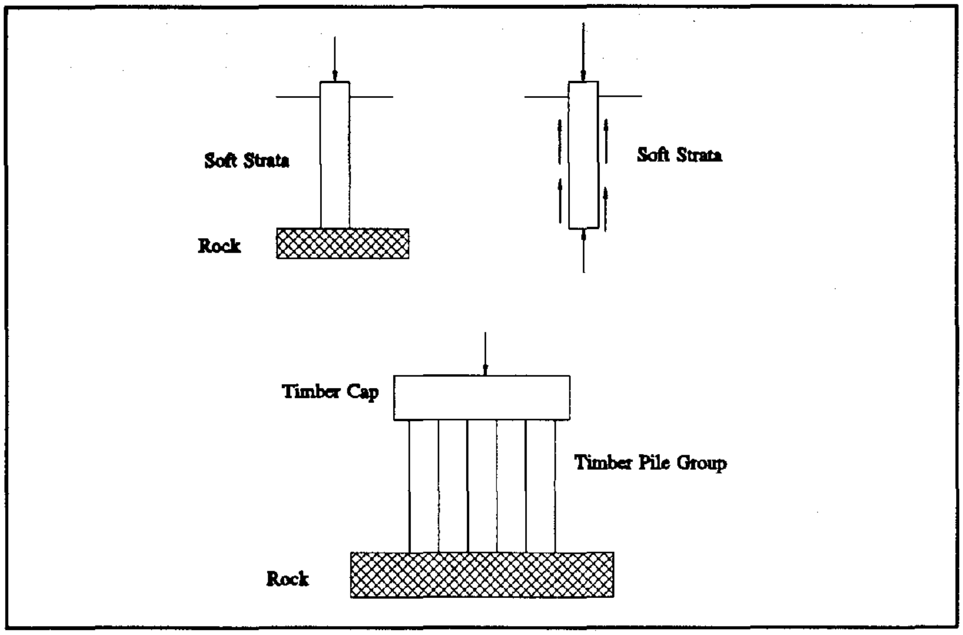

Figure 3. Pile foundations.

No set of simple rules or procedures can be expected to cover the entire variety of conditions that might affect pile foundations. The following discussion outlines some basic design criteria that a design engineer may find useful in meeting the basic requirements of safety and economy. As an example, a pile foundation design guideline is presented, followed by a design example to illustrate a specific site condition where piles are needed.

Design Summary

The bearing capacity of a single pile is controlled by the structural strength of the pile and the strength of the soil, and the smaller of the two values should be used for design:

(1) Pile capacity as determined by the structural strength of pile.

- To avoid overstress in the pile under design load, the allowable pile load-carrying capacity can be calculated from the allowable design values given in

AASHTO table 4.5.7.3.A. The design value is multiplied by the area of the pile's critical section.[12]

(2) Pile capacity as determined by the supporting strength of the soil.

The design of pile foundations based on static formulas is site specific, and for illustrative purposes, a site with a cohesive soil (clay) will be used to present a typical pile design. The design methodology presented in this section is based primarily on guidelines given by references 11 and 13. The ultimate load-carrying capacity of a pile can be taken simply as the sum of the load carried at the pile tip plus the frictional resistance (skin resistance):[11]

$$Q_u = Q_p + Q_s \tag{1}$$

where,

Q_u = Ultimate pile capacity
Q_p = Load-carrying capacity of pile tip
Q_s = Frictional resistance

- Tip-bearing capacity, Q_p

The static pile formulae used to approximate the bearing capacity of piles driven on or into a hard stratum or soft rock is:

$$Q_p = \pi\ R^2\ (\ 1.3\ c\ N_c + \gamma\ D\ N_q + 0.6\ \ R\ N_\gamma\ ) \tag{2}$$

where,

Q_p = Ultimate bearing capacity of single pile
R = Radius of pile tip
c = Cohesion of soil
γ = Unit weight of soil
D = Total penetration of pile, from ground surface to pile tip
N_c, N_q, N_γ = Bearing capacity factors

For piles in saturated clays (the soil friction angle ϕ is equal to 0.0), the bearing capacity can be simplified to:[11]

$$Q_p = 9\ c_u\ A_p \tag{3}$$

where,

Q_p = Load-carrying capacity of pile point
c_u = Undrained cohesion of the soil below pile tip
A_p = Area of pile tip

- Frictional resistance, Q_s

The frictional resistance of a pile in clay is equal to the unit adhesion multiplied by the embedded length area of the pile. There are various methods for calculating the frictional resistance, and for illustrative purposes it can be taken as:[11]

$$Q_s = \Sigma\ \alpha\ c_u\ p\ \Delta L \tag{4}$$

where,

Σ = Summation of contributions from several strata
α = Empirical adhesion factor = 1.0 for normally consolidated clays
c_u = Undrained cohesion of the soil below pile tip
p = Perimeter of pile section
ΔL = Pile length

- Allowable pile capacity, Q_{all}

Once the ultimate pile capacity is found by adding the point-bearing capacity, Q_p, and the frictional capacity, Q_s, a reasonable factor of safety should be used to determine the total ultimate load-carrying capacity.[11]

$$Q_{all} = \frac{Q_u}{FS} \tag{5}$$

where,

Q_{all} = Allowable load-carrying capacity
Q_u = Ultimate pile capacity (equation 1)
FS = Factor of safety = 1.9 to 3.5 (AASHTO table 4.5.6.2A)[12]

Pile Spacing and Group Action

In most cases, pile foundations are constructed as groups of closely spaced piles joined by a continuous cap. The spacing of piles is such that the bearing capacity of the group is not less than the sum of the bearing capacities of the individual piles. For practical purposes, a lower limit of pile spacing is taken as 2.5 times the pile diameter.[12]

DESIGN EXAMPLE

A stress-laminated timber bridge deck is to be constructed at a site with cohesive soil deposits. The design conditions and assumptions are as follows:

ϕ = Soil friction angle = 0

c = Soil cohesion = 1,100 lbf/ft^2 (52.69 kN/m^2) (Typical value for soil cohesion below pile tip in saturated clay. For other conditions, see references 10 and 13)
D = Pile diameter = 12 in (304.8 mm) taken at 3 ft (0.915 m) from butt
ΔL = Pile length = 40 ft (12.2 m)
Pile material = Douglas fir timber
HS-20 AASHTO loading, two-lane stress-laminated timber bridge
Span length = 20 ft (6.1 m)
Width = 24 ft (7.32 m)
Deck thickness = 14 in (355.6 mm)

Dead Load and Live Load Calculations

The details of the design of this bridge deck are presented in reference 14.

The magnitudes of the total dead load and live loads are:

Total equivalent dead load = 48,184 lbf (214 kN)

For this bridge span and a single-lane HS-20 AASHTO loading, the end reaction = 41,600 lbf (185 kN) (see figure 4).
Total live load = 41,600 lbf x 2 = 83,200 lbf (370 kN)
Total load = 83,200 lbf + 48,184 lbf/2 = 107,292 lbf (476 kN) = 53.6 tons

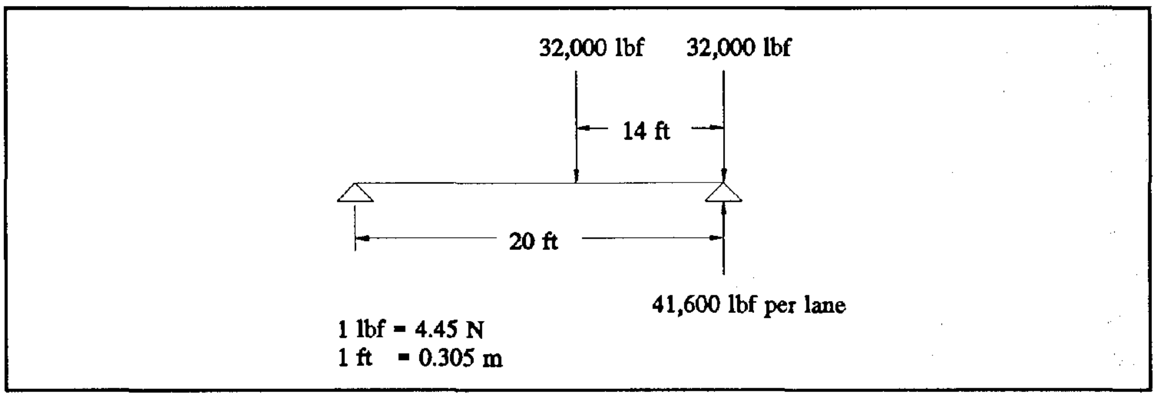

Figure 4. Live load.

Pile capacity as determined by the structural strength of the pile

- The allowable pile load-carrying capacity can be calculated from the allowable design values given in AASHTO table 4.5.7.3A, multiplied by the area of the critical section of the pile.

  Usually, the critical section at 9 to 10 in (228 mm to 254 mm) diameter controls the design capacity of friction piles, and the area of the tip controls the design of end-bearing piles.[15]

  From AASHTO table 4.5.7.3.A, the allowable compression parallel-to-grain working stress for douglas fir is 1,200 lbf

/in^2 (8.2 MPa). The area at the critical section, where the diameter d is assumed to be 9 in (228 mm), becomes

$$A = \frac{\pi \times d^2}{4} = \frac{\pi \times 9^2}{4} = 63.6\,in^2\,(41{,}022\,mm^2) \tag{6}$$

Then, the allowable load on a pile is

$$Q_{all} = \frac{1{,}200\ lb/in^2 \times 63.6\ in^2}{2{,}000\ lb/ton} = 38\ tons\ (34.5\,metric\ tons) \tag{7}$$

Pile capacity as determined by supporting strength of the soil

$$Q_u = Q_p + Q_s \tag{8}$$

End-bearing capacity:

$$Q_p = 9\ c_u\ A_p \tag{9}$$

Diameter at the tip from ASTM D25-91 table 1 = 6.4 in (162.5 mm); therefore, area of pile tip is

$A_p = \pi \times (6.4\ \text{in})^2 / 4 = 32.15\ \text{in}^2 = 0.22\ \text{ft}^2\ (.02\ \text{m}^2)$

The undrained cohesion value is
$c_u = 1{,}100\ \text{lbf/ft}^2\ (53\ \text{kN/m}^2)$

$$Q_p = \frac{9 \times 1{,}100\ lb/ft^2 \times 0.22\ ft^2}{2{,}000\ lb/ton} = 1.09\,tons\,(1\,metric\ ton) \tag{10}$$

Frictional resistance:

$$Q_s = \alpha \ c_u \ p \ \Delta L \tag{11}$$

where,

α = 1.0

p = perimeter at the critical section where d = 9 in (228 mm)

= 2 x π x r = 2 x 3.14 x 9 in / 2 = 28.26 in = 2.35 ft (0.7 m)

Required pile length, ΔL = 40 ft (12.2 m)

$$Q_s = \frac{1 \times 1{,}100 \ lb/ft^2 \times 2.35 \ ft \times 40 \ ft}{2{,}000 \ lb/ton} = 51.7 \ tons \ (51 \ metric \ tons) \tag{12}$$

Allowable load:

$$Q_{all} = \frac{Q_u}{FS} \tag{13}$$

where,

Q_u = 1.09 tons + 51.7 tons = 52.79 tons (48 metric tons)

FS = 3.5

$$Q_{all} = \frac{52.79 \ tons}{3.5} = 15 \ tons \ (13.6 \ metric \ tons) \tag{14}$$

$$No. \ of \ piles = \frac{Q_{app}}{Q_{all}} \tag{15}$$

$$No. \ of \ piles = \frac{53.6 \ tons}{15 \ tons} = 3.57 \ piles \tag{16}$$

Use five piles.

The structural capacity of the pile material is 38 tons (34.5 metric tons). However, the allowable pile capacity as determined by the supporting strength of the soil is 15 tons (13.6 metric tons); therefore the design is OK.

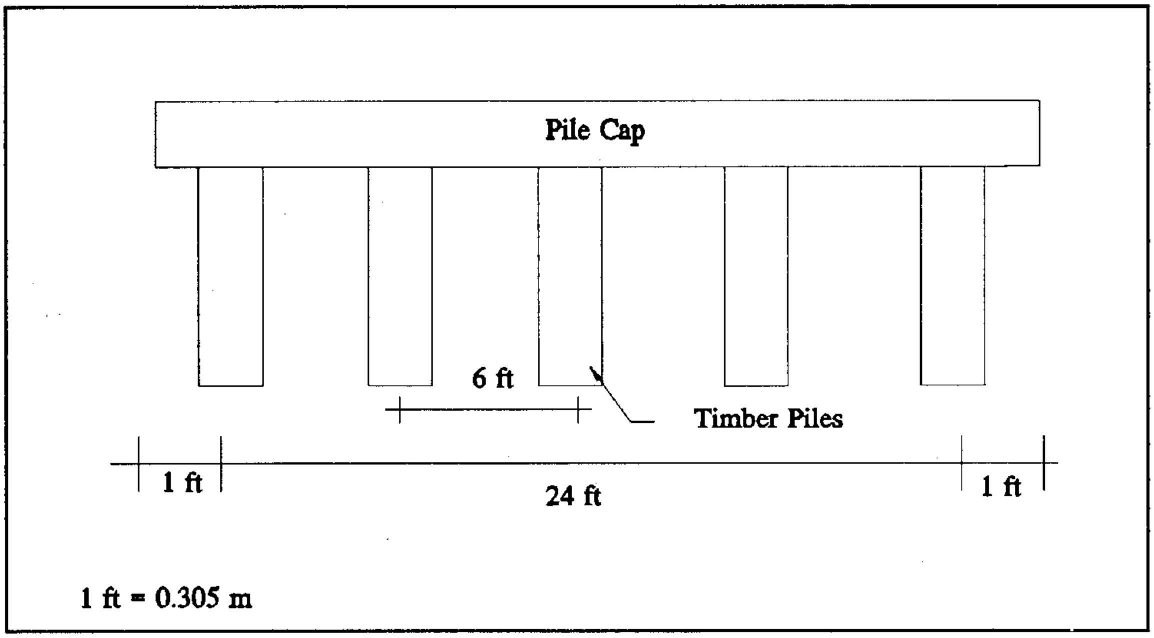

Figure 5. Timber piles and pile cap.

Timber Cap Design

Timber Cap Dimensions: 16 in by 16 in by 26 ft (406 mm by 406 mm by 7.9 m) (see figure 5).

Material Properties:

Douglas fir-larch grade #1. The allowable design values given in NDS table 4D (NDS 1991) are:[16]

Bending strength:
$$F_b' = F_b \times C_m \times C_F = 1{,}200 \text{ lbf/in}^2 \times 1.0 \times 0.98 = 1{,}176 \text{ lbf/in}^2 \text{ (8 MPa)}$$

Compression perpendicular to grain:
$$F_{cp}' = F_{cp}' \times C_m = 625 \text{ lbf/in}^2 \times 0.67 = 419 \text{ lbf/in}^2 \text{ (2.8 MPa)}$$

Shear strength:
$$F_v' = F_v \times C_m = 85 \text{ lbf/in}^2 \times 1.0 = 85 \text{ lbf/in}^2 \text{ (0.57 MPa)}$$

where,
C_m = Moisture content factor
C_F = Size factor

Dead and live load

Maximum live and dead loads on each abutment.

Dead load of superstructure = 48,184 lbf (214 kN)
Weight of cap = (16 in/12 ft) x (16 in/12 ft) x 26 ft x 58 lbf/ft^3
= 2,680 lbf (11.7 kN)
Total dead load on each abutment: (48,184 lbf/2) + 2,680 lbf = 26,772 lbf (119 kN)
Total dead load on pile-line per unit width: 26,772 lbf / 24 ft = 1,115 lbf/ft (16 kN)

Live load on abutment:

The distribution width, D is:[3]

$$D = [\, b_f + (2 \times d)\,] \times C_b \qquad \textbf{(17)}$$

where,

D = Transverse wheel load distribution width in inches (mm).
P_w = Wheel load = 16,000 lbf (71.2 kN)

b_f = Truck tire width perpendicular to traffic = $\sqrt{0.025\, P_w}$ in (mm).
= $(0.025P_w)^{0.5}$ = $(0.025 \times 16{,}000 \text{ lbf})^{0.5}$ = 20 in (508 mm)
d = Deck thickness = 14 in (356 mm)
C_b = Butt joint factor
Butt joint pattern: 1 in 5

$$C_b = \frac{j}{j+1} \qquad \textbf{(18)}$$

where,

j = Number of laminae between two successive butt joints = 4
C_b = 4/5 = 0.8
D = (20 in + 28 in) x 0.8 = 38.4 in (975 mm)

The live load is distributed over a portion of the deck equal to 38.4 in = 3.2 ft (0.97 m). The live load for a single lane is 20,800 lbf (92.5 kN) (AASHTO appendix A).
Live load: 20,800 lbf/ 3.2 ft = 6,500 lbf/ft (95 kN/m)

Designing for a critical section of the pile cap (figure 6):

$R_A = R_B$ = [(1,115 lbf/ft x 6 ft) + (6,500 lbf/ft x 3.2 ft)] / 2 = 13,745 lbf (58 kN)

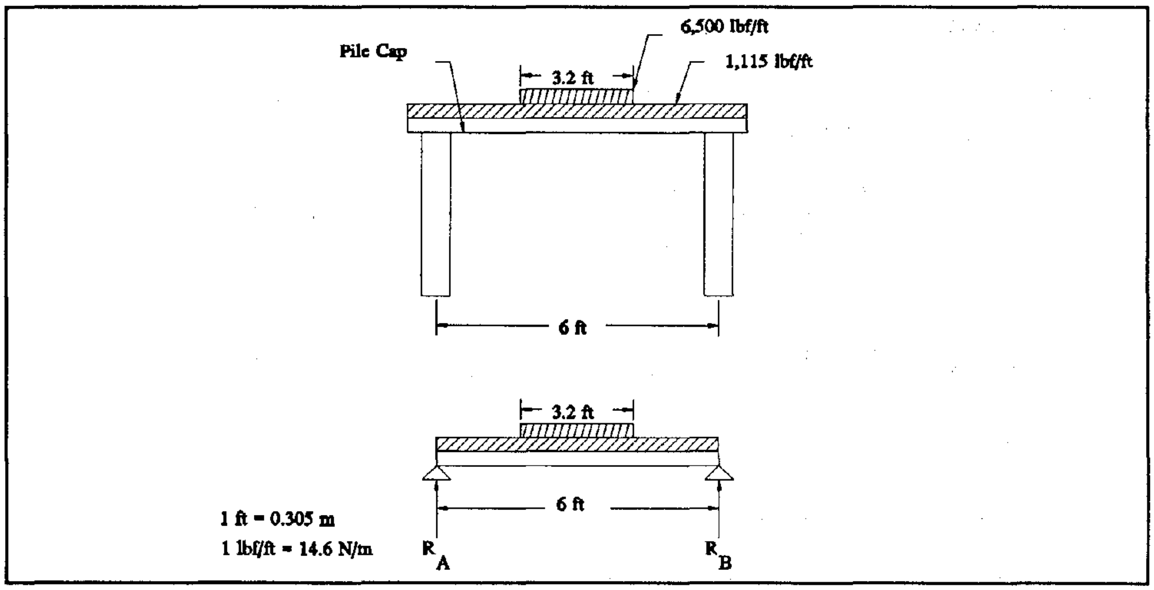

Figure 6. Pile section.

Horizontal shear:

Horizontal shear stress is computed by evaluating the live load vertical shear at the lesser distance of L/4 or 3d.[12]

$$\frac{6\,ft}{4} = 1.5\,ft, \;\; 3 \times 14\,in/12\,ft = 3.5\,ft\ (1.06\,m) \tag{19}$$

Therefore, the distance of 1.5 ft (1.06 m) controls the design for horizontal shear.

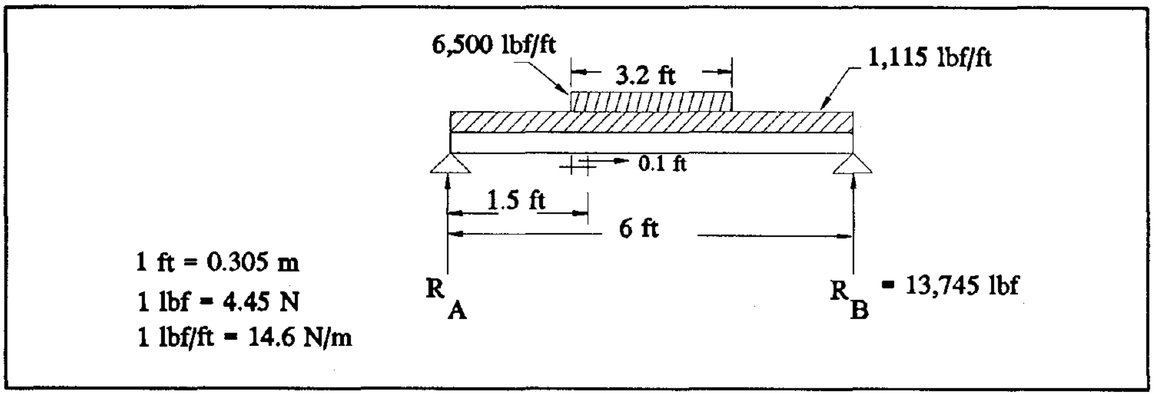

Figure 7. Vertical shear.

From figure 7:

V = 13,745 lbf - [(1.5 ft x 1,115 lbf/ft) + (0.1 ft x 6,500 lbf/ft)]
= 11,422.5 lbf (49 kN)

$$f_v = \frac{3 \times V}{2b \times d} = \frac{3 \times 11{,}422.5\ lb}{2 \times 16\ in \times 16\ in} = 67\ lbf/in^2\ (0.46\ MPa) \qquad \textbf{(20)}$$

f_v = 67 lbf/in^2 (0.46 MPa) < F_v' = 85 lbf/in^2 (0.57 MPa) OK

Bending stress:

By symmetry (figure 8), the reaction R_1 is

R_1 = [(6,500 lbf/ft x 3.2 ft) / 2] = 10,400 lbf (46.2 kN)

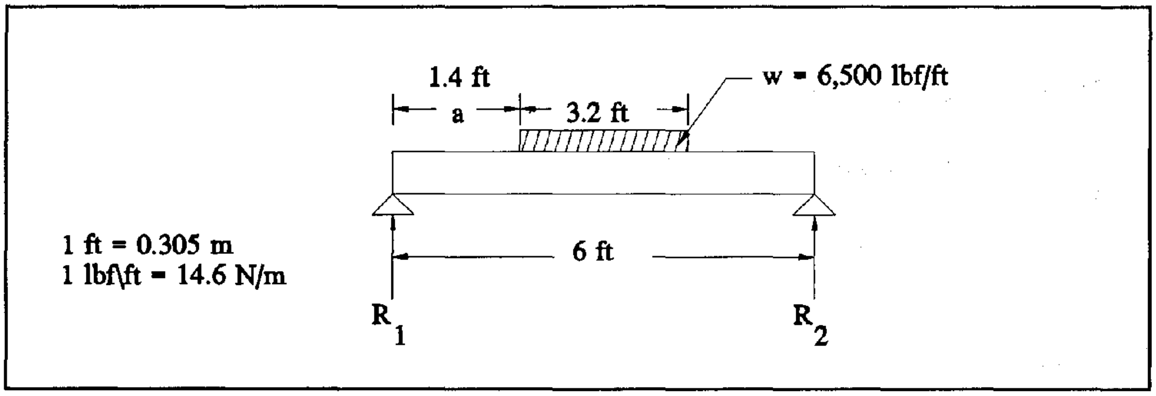

Figure 8. Live load moment.

The maximum live load moment at midspan is

$$(M_{\max})_{LL} = R_1 \left(a + \frac{R_1}{2 \times w} \right) \qquad \textbf{(21)}$$

(For equation 21, see AISC manual, 2-111).[17]

$$(M_{\max})_{LL} = 10{,}400\,lbf\ (1.4 + \frac{10{,}400\ lbf}{2 \times 6{,}500\ lbf/ft}) = 22{,}880\ lb\text{-}ft\ (31\,kN\text{-}m) \qquad \textbf{(22)}$$

The maximum dead load moment at midspan is

$$(M_{\max})_{DL} = \frac{w \times L^2}{8} = \frac{1{,}115\ lbf/ft \times (6 ft)^2}{8} = 5{,}018\ lbf\text{-}ft\ (7\ kN\text{-}m) \tag{23}$$

Total moment in cap = 22,880 lbf-ft + 5,018 lbf-ft = 27,898 lbf-ft (38 kN-m)

For continuity of supports use a factor of 0.8.

Therefore, M_{max} = 0.8 x 27,898 lbf-ft = 22,318 lbf-ft (30 kN-m)

The bending stress is

$$f_b = \frac{M_{\max}}{S} \tag{24}$$

where,
$S = bd^2/6 = 16 \times 16^2/6 = 683\ in^3\ (0.01\ m^3)$

Therefore, f_b becomes

$$f_b = \frac{22{,}318\ lbf\text{-}ft \times 12\ in/ft}{683\ in^3} = 392\ lbf/in^2\ (2.7\ MPa) \tag{25}$$

$f_b = 392\ lbf/in^2\ (2.7\ MPa) < F_b' = 1{,}176\ lbf/in^2\ (8\ MPa)$ OK

Bearing:

$F_{cp}' = 417\ lbf/in^2\ (2.87\ MPa)$

$$Required\ Bearing\ Area = \frac{R}{F_{cp}'} = \frac{13{,}745\ lbf}{417\ lbf/in^2} = 33\ in^2\ (21{,}285\ mm^2) \tag{26}$$

$$Required\ width = \frac{33\ in^2}{38.4\ in} = 0.86\ in\ (22\ mm) \tag{27}$$

0.86 in (22 mm) < 16 in (406 mm) provided by cap OK

The details of the design are summarized in figure 9.

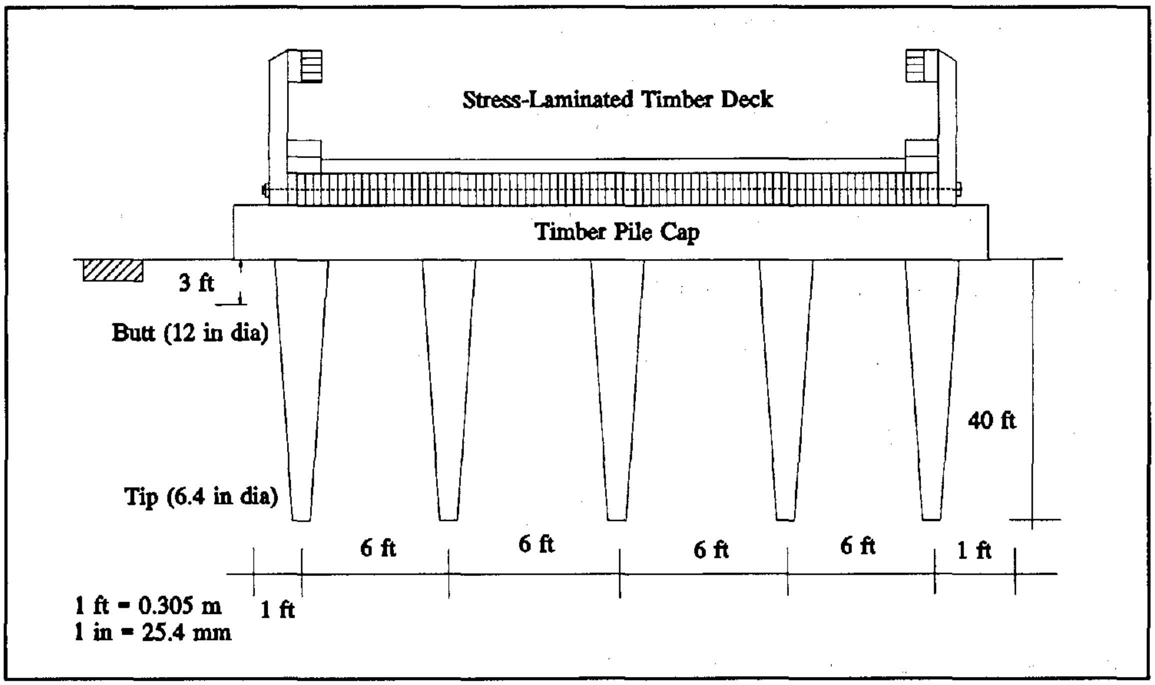

Figure 9. Timber piles for bridge abutment.

AN OVERVIEW OF STUB ABUTMENTS

Stub abutment systems can be easily used at locations where the rock formations, with moderately high bearing capacity, are found at shallow depths, which is commonly encountered in West Virginia. In addition, stub abutments can be built on pile foundations should the soil under the footing exhibit excessive settlement. A typical cross section of a timber stub abutment is shown in figure 10. The superstructure is supported on short timber columns that are connected to a concrete spread footing.

Due to the shallow abutment heights, the manufacturing, transportation, and erection costs of stub abutments may be less than those of conventional abutments. However, when stub abutments are used, the initial cost of the superstructure can be higher due to the increase in the bearing-to-bearing span length of the bridge.

Timber backing planks are attached to the timber columns with galvanized lag screws to retain the backfill material. Stub abutments are generally designed as gravity walls, and a stability analysis should be performed following the procedures outlined in chapter 5 of this booklet. Although no specific design information on timber stub abutments is available, the guidelines given for concrete and steel stub abutments in reference 37 can be used for further information.

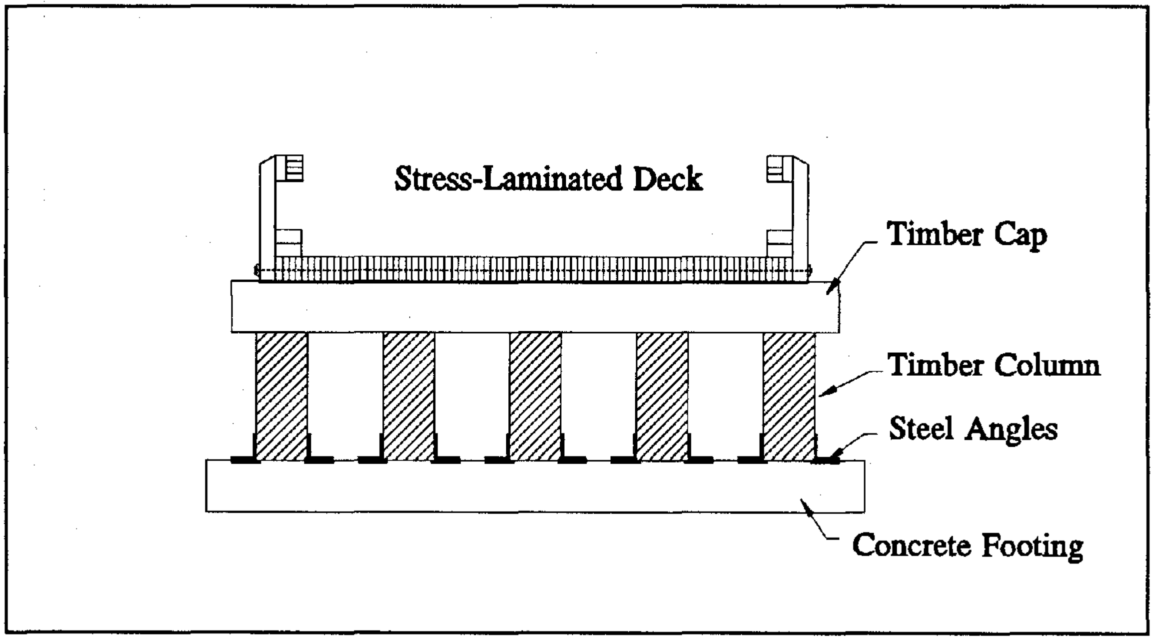

Figure 10. Stub abutment.

CHAPTER 3. STEEL BENT AND DRIVEN STEEL PILE ABUTMENTS WITH TIMBER LAGGING

OVERVIEW

Elevation and plan views of a steel bent abutment system and a driven steel pile bent system are shown in figures 11 and 12. Timber backing planks are attached to the H-piles with galvanized metal clips that grip the planks against the flange of the piles. A timber cap is attached to the steel H-piles by welding a steel plate to the top of the pile and bolting the timber to the plate. In steel bent systems, the end of the column is welded to a base plate that is recessed into the concrete footing and connected to it with high-strength bolts. To prevent corrosion of base plates or high-strength bolts, the recess is filled with non-shrink or expandable grouts. The recess would also act as a shear transfer device in case the bolted connection fails due to corrosion over a period of time. This chapter presents two substructure designs of steel sections with timber lagging for short span bridges. For further information on driven piles, see references 39 and 40 listed at the end of this manual.

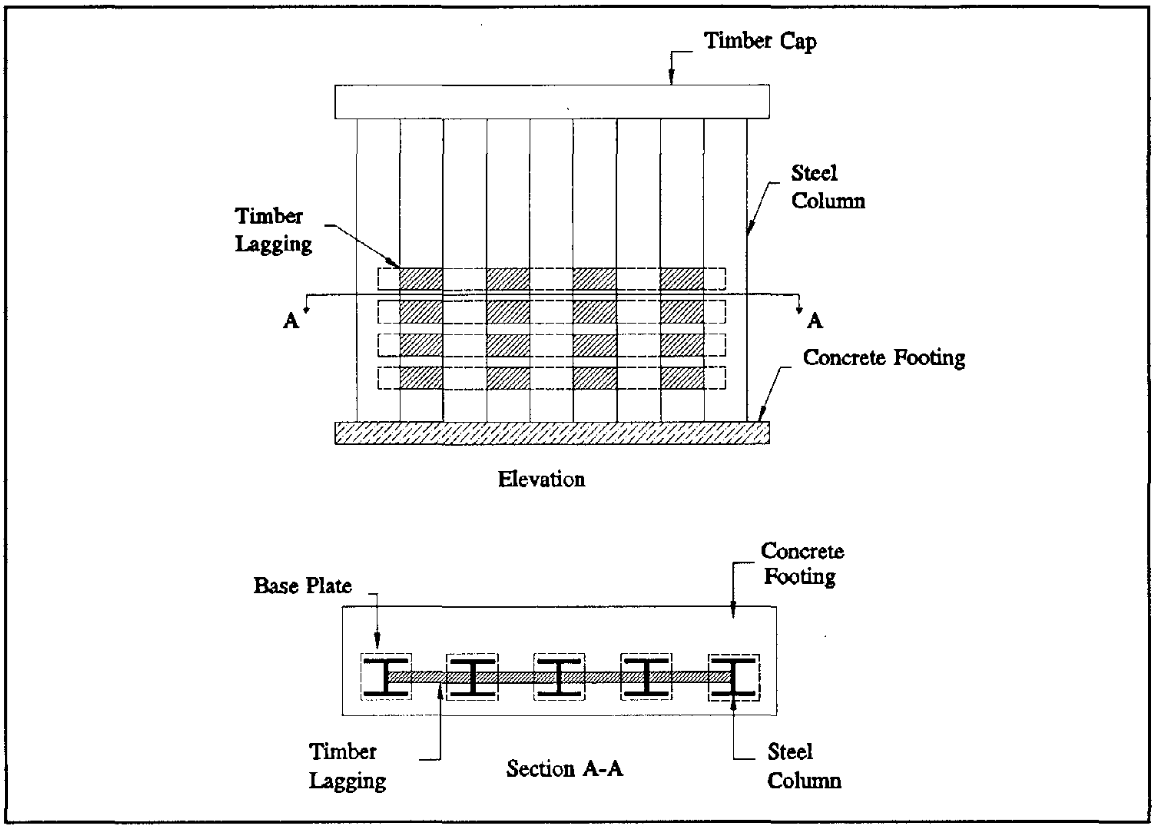

Figure 11. Steel bent abutment.

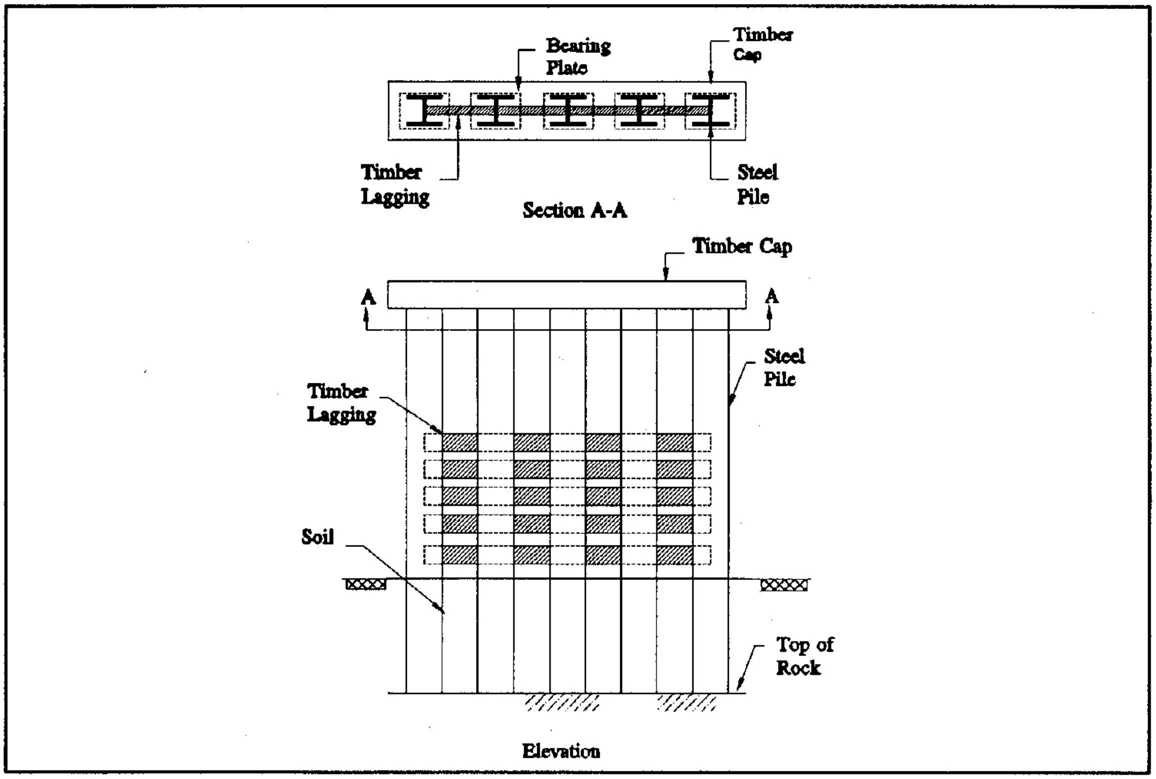

Figure 12. Driven steel pile abutment.

DESIGN PROCEDURE

Since the superstructure is placed directly over the pile cap, the axial load on the steel columns is equal to the dead and live loads of the superstructure. It is assumed that the superstructure is pin-connected to the pile cap, and therefore, the steel columns are designed for combined stresses due to axial loads and bending loads from soil pressure. The design checks for overturning, and the computations for sliding and bearing capacity are similar to the ones used for any abutment design. The pile bents must be designed for longitudinal forces, wind loads, and overturning pressures. The design of driven steel pile bent systems presented in this chapter follows the guidelines given in the *Highway Structures Design Handbook* of the U.S. Steel Corporation.[18] The following general design procedure for steel-driven pile abutments is suggested:

1. Determine the vertical and horizontal loads per pile. Determine the wind loads in the x and y directions and longitudinal traction forces in the y-direction, and compute the resultant forces parallel to the row of piles (H_x) and normal to the row of piles (H_y), as shown in figure 13. A single row of piles is used. Determine the vertical load per pile by taking moments about the top of the piles and applying the following equation:[19]

$$Q_m = \frac{F_v}{n} \pm \frac{M_y x}{\sum x^2} \qquad \textbf{(28)}$$

where,

Q_m	=	Vertical load on any given pile, m
F_v	=	Total vertical load due to dead load and live load acting at the centroid of the pile group, lbf (N)
M_y	=	Moment with respect to the y-axis through the centroid of the pile group, lbf-ft (N-m)
x	=	Distance from a given pile m to the y-axis, ft (m)
n	=	Number of piles

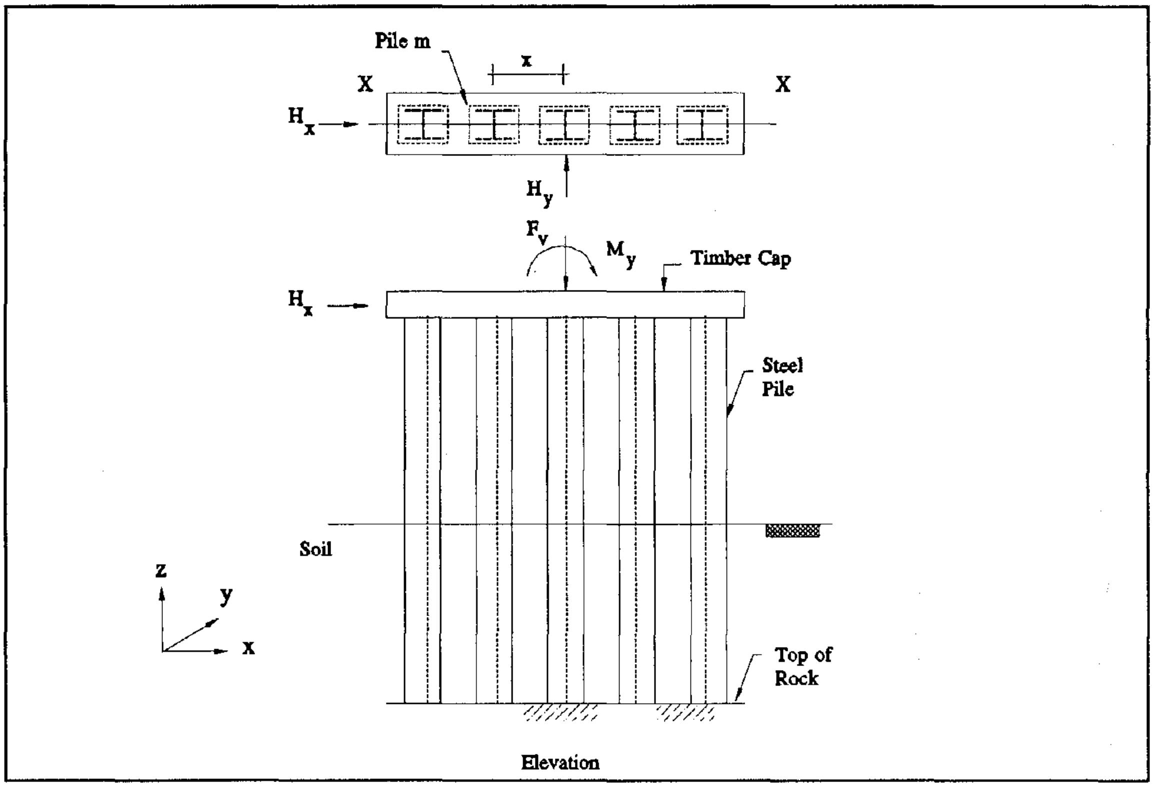

Figure 13. Definition of terms used for pile bents.

2. Tentatively choose the pile size and determine the depth of penetration required to develop the axial load in the pile. If the pile extends above the ground surface, combined stresses due to axial and bending loads may govern the pile size.

3. Determine the depth below ground surface (depth of fixity) at which the piles may be considered fixed. A partially embedded pile is required to carry a

vertical load in addition to a horizontal load and a bending moment. In order to compute allowable stresses to avoid buckling, a free-standing column fixed at a depth D is assumed. The required depth of fixity for a partially embedded pile is given by

$$D = 1.8 \left[\frac{EI}{n_h} \right]^{1/5} \tag{29}$$

where,

D	= Embedded length
E	= Modulus of elasticity of pile
I	= Moment of inertia for pile
n_h	= Constant of horizontal subgrade reaction

For further guidance on the depth of fixity, see references 13, 20, and 21.

4. Determine the allowable compressive stress in the pile using the approximate column formula given by AASHTO table 10.32.1A.[12]

5. Determine the maximum stresses in the pile due to axial and bending loads according to AASHTO article 10.36.[12]

DESIGN EXAMPLE I: STEEL BENT ABUTMENT

This example presents the design of a steel bent abutment (as shown in figure 11 or 12) to support a stress-laminated timber bridge deck.

Assumptions and Design Data

Deck material:	Northern Red Oak
Bridge span:	20 ft (6.1 m)
Bridge width:	24 ft (7.3 m)
Loading:	HS-20
Lanes:	2
Timber cap:	Douglas fir-larch (16 in x 16 in x 26 ft) Specific weight = 50 lbf/ft^3 (7.85 kN/m^3)
Number of columns:	5
Spacing between columns:	6 ft (1.83 m)
Height of column:	18 ft (5.5 m)
Total height of abutment:	19.33 ft (5.9 m) (column height + timber cap depth)
K_a = Coefficient of active earth pressure:	0.3
γ_s = Unit weight of soil:	100 lbf/ft^3 (15.7 kN/m^3)

Live load surcharge (AASHTO 3.20.3): 2 ft (0.61 m)
Steel grade: ASTM A36

1. **Determine the vertical and horizontal load per pile.**

I. Total dead load of timber deck:[14]

48,184 lbf (214 kN)

II. Live load (see figure 4):

41,600 lbf x 2 = 83,200 lbf (370 kN) (AASHTO appendix A)

III. Timber cap:

46.2 ft^3 x (50 lbf/ft^3 + 8 lbf/ft^3 "creosote retention") = 2,680 lbf (12 kN)

IV. Total (dead + live) loads on single abutment:

$$\frac{48{,}184\ lbf}{2} + 83{,}200\ lbf + 2{,}680\ lbf = 109{,}972\ lbf\ (489\ kN) \tag{30}$$

V. Computation of soil pressure:

Soil pressures on the abutments are computed on the basis of the pressure distribution shown in figure 14.

Uniform vertical pressure due to surcharge, q = 2 ft x 100 lbf/ft^3 = 200 lbf/ft^2
= (9.4 kN/m^2)

Earth pressure resultant due to surcharge = q x K_a x H
= 200 lbf/ft^2 x 0.3 x 18 ft
= 1,080 lbf/ft (16 kN/m) of width

Earth pressure resultant due to backfill soil = 1/2 x γ_s x H^2 x K_a
= 1/2 x 100 lbf/ft^3 x $(18\ ft)^2$ x 0.3
= 4,860 lbf/ft (71 kN/m) of width

Total soil pressure resultant for one column due to surcharge:

6 ft x 1,080 lbf/ft = 6,480 lbf (29 kN)

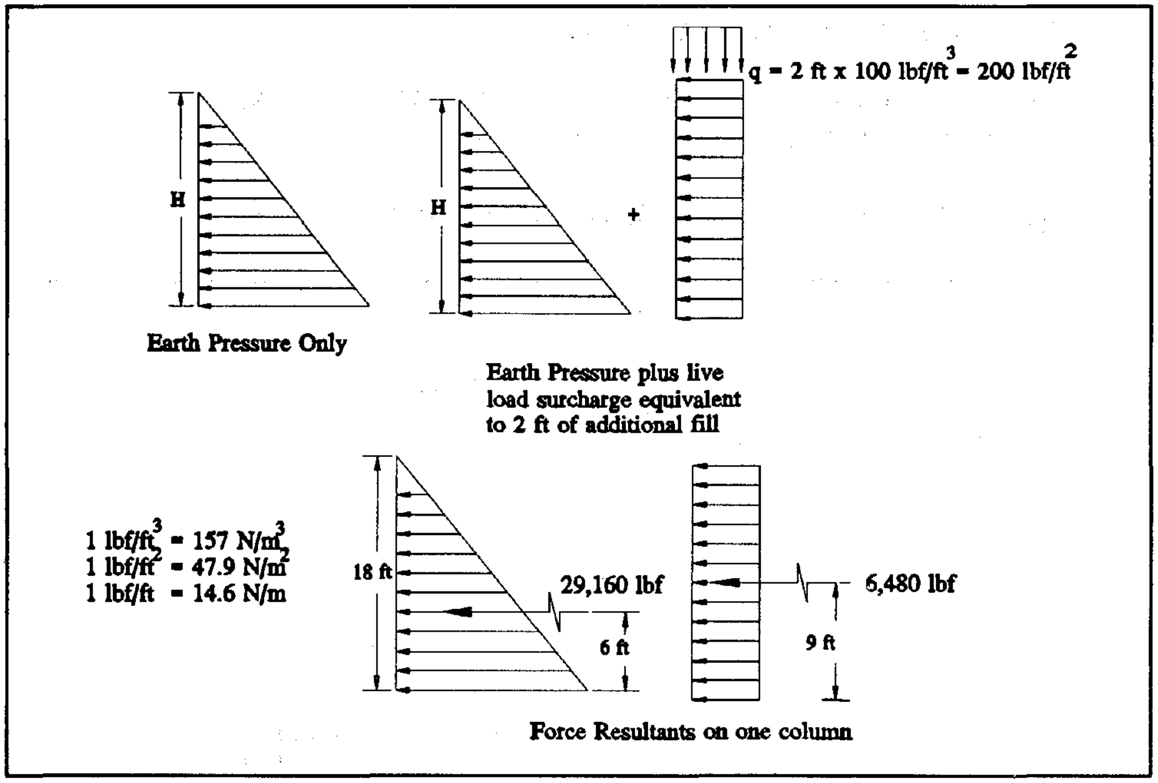

Figure 14. Soil pressures.

Total soil pressure resultant for one column due to backfill:

6 ft x 4,860 lbf/ft = 29,160 lbf (129 kN)

VI. *Impact load on abutment*:

Impact is not considered in accordance with AASHTO article 3.8.1.2.

VII. *Longitudinal forces:*

Based on vehicle loads, AASHTO article 3.9 specifies an approximate longitudinal force equal to 5 percent of the live load applied in all lanes carrying traffic in the same direction. The live load used to compute the longitudinal force is the uniform lane load specified by AASHTO as 640 lbf/ft (9.3 kN/m) plus the concentrated lane load for moment, which is 18,000 lbf (80 kN) according to AASHTO article 3.7.1.[12] The longitudinal force is applied 6 ft (1.83 m) above the deck surface.

Longitudinal force = (640 lbf/ft x 20 ft + 18,000 lbf) x 0.05 x 2 = 3,080 lbf
= (13.7 kN)

VIII. Total forces on column (figure 15):

Vertical live and dead loads = 109,972 lbf/5 = 22,000 lbf (98 kN)
Total vertical load on column = 22,000 lbf (97.8 kN)

Longitudinal force per column = 3,080 lbf/5 = 620 lbf (2.8 kN)

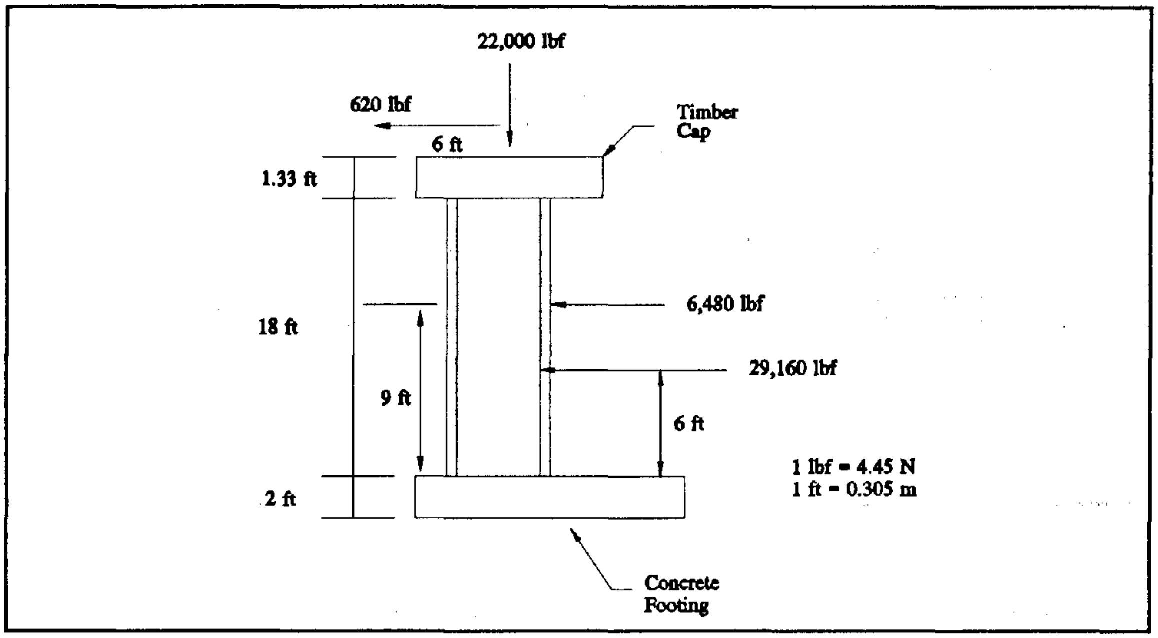

Figure 15. Forces on column.

2. Tentatively choose pile size.

Try W 14 x 109

From the *Manual of Steel Construction* (AISC), the section properties are:

d = 14.32 in (363 mm)	S_x = 173 in^3 (2,832 mm^3)	A = 32 in^2 (20,640 mm^2)
t_w = 1/2 in (12.7 mm)	r_x = 6.22 in (158 mm)	r_y = 3.78 in (96 mm)

3. Determine the maximum stresses in the pile due to axial and bending loads according to AASHTO article 10.36.[12]

According to AASHTO article 10.36, all members subjected to both axial compression and bending stress shall be proportioned to satisfy the following requirement:

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - \frac{f_a}{F'_{ex}}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - \frac{f_a}{F'_{ey}}) F_{by}} \leq 1 \tag{31}$$

where,

$$F'_e = \frac{\pi^2 E}{F.S. \ (K_b L_b / r_b)^2} \tag{32}$$

f_a	=	Computed axial stress
f_{bx} or f_{by}	=	Computed compressive bending stress about the x-axis and y-axis
F_a	=	Axial stress that would be permitted if axial force alone existed, regardless of the plane of bending
F_{bx} , F_{by}	=	Compressive bending stress that would be permitted if bending moment alone existed about the x-axis and the y-axis, respectively, as evaluated according to AASHTO table 10.32.1A
F'_e	=	Euler buckling stress divided by a factor of safety
E	=	Modulus of elasticity of steel
K_b	=	Effective length factor in the plane of bending (see AASHTO appendix C)
L_b	=	Actual unbraced length in the plane of bending
r_b	=	Radius of gyration in the plane of bending
C_{mx} , C_{my}	=	Coefficient about the x-axis and y-axis, respectively, whose value is taken from AASHTO table 10.36A = 0.85
F.S.	=	Factor of safety = 2.12

However, should bending occur about one axis only, the appropriate term for the other axis will drop out of the equation. Since bending occurs about the x-axis only, equation 31 becomes

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - \frac{f_a}{F'_{ex}}) F_{bx}} \leq 1 \tag{33}$$

The axial stress is:

f_a = P/A = 22,000/32 = 687.5 lbf/in^2 (4.72 MPa)

The boundary conditions shown in figure 16 are assumed (conservative), and the maximum moment, $(M_x)_{max}$, is computed at the base of the column, from which the bending stress is computed as:

$$f_{bx} = \frac{(M_x)_{max}}{S_x} \tag{34}$$

$$f_b = \frac{[29{,}160\ lbf \times 6\ ft + 6{,}480\ lbf \times 9\ ft + 620\ lbf \times 25.33\ ft] \times 12\ in/ft}{173\ in^3} = 17{,}270\ lbf/in^2\ (119\ MPa) \tag{35}$$

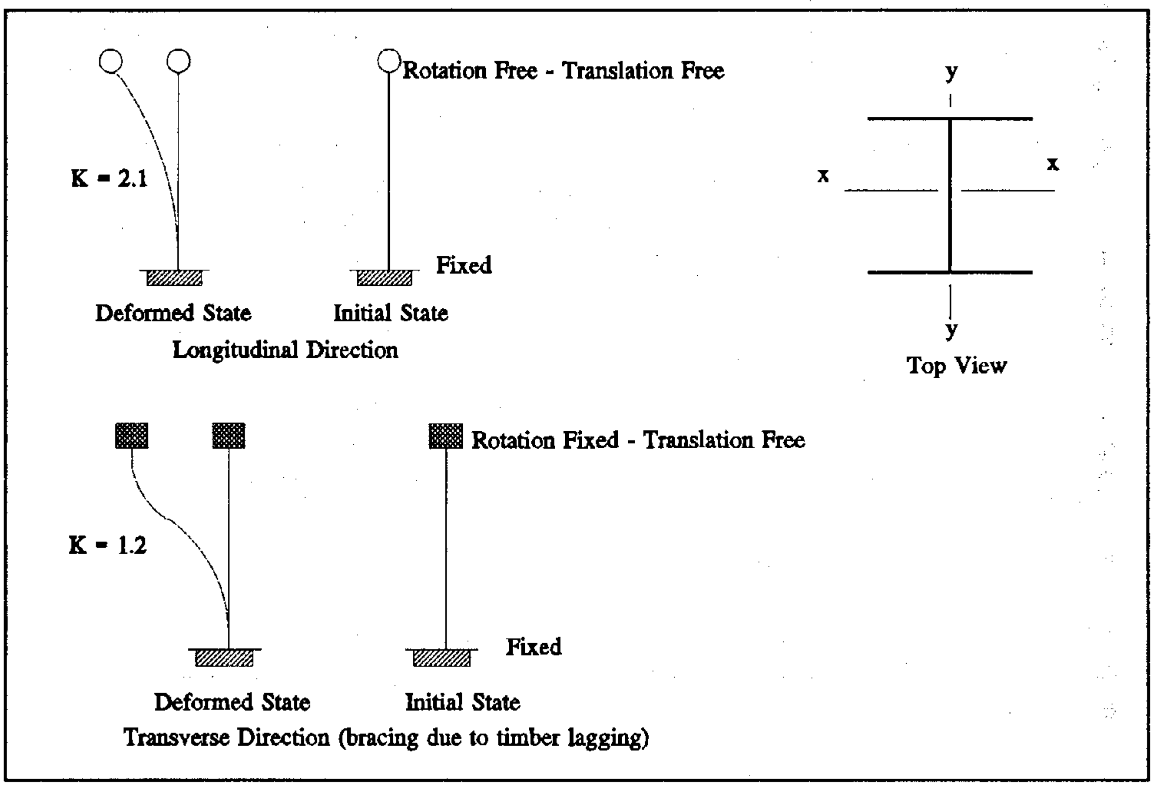

Figure 16. Assumed boundary conditions.

Based on figure 16, the effective length factor in the longitudinal direction becomes K_x=2.1. In the transverse direction, the column can be assumed fixed at the base and free to translate at the top, and therefore, the effective length factor K_y = 1.2.

In the longitudinal direction, the column slenderness ratio for bending about the x-axis is $K_x\ L_x/r_x$, and in the transverse direction, that is bending about the y-axis, the slenderness ratio is $K_y\ L_y/r_y$. Neither of these two ratios should be greater than $C_c = (2\pi^2 E/F_y)^{1/2} = 126.1$ (AASHTO 10.32.1A), and the largest slenderness ratio controls.

$$\frac{K_x L_x}{r_x} = \frac{2.1 \times 18\,ft \times 12\,in/ft}{6.22\,in} = 73 < C_c = 126, \rightarrow Controls \tag{36}$$

$$\frac{K_y L_y}{r_y} = \frac{1.2 \times 18\,ft \times 12\,in/ft}{3.73\,in} = 69.5 < C_c = 126 \tag{37}$$

The basic allowable axial strength, F_a, is 16,980 lbf/in^2. This value is reduced by $0.53\ (K_xL_x/r_x)^2$. Therefore,

$F_a = 16{,}980 - 0.53\ (K_xL_x/r_x)^2 = 16{,}980$ lbf/in^2 - $0.53\ (73)^2 = 14{,}155$ lbf/in^2 (97 MPa)

f_a/F_a = 687.5 lbf/in^2 / 14,155 lbf/in^2 = 0.0486
F_{bx} = 20,000 lbf/in^2 (2,756 MPa) (AASHTO table 10.32.A)

For bending, the slenderness ratio in the plane of bending should be used; in this case K_xL_x/r_x.

$$F'_{ex} = \frac{\pi^2 \times 29 \times 10^6}{2.12\ (73)^2} = 25{,}334\ lbf/in^2\ (174\ MPa) \tag{38}$$

Substituting in equation 33,

$$0.0486 + \frac{0.85 \times 17{,}270\ lbf/in^2}{(1 - \frac{687.5\ lbf/in^2}{25{,}334\ lbf/in^2})20{,}000\ lbf/in^2} = 0.0486 + 0.75 = 0.80 < 1\ \ OK \tag{39}$$

Therefore, W 14 x 109 is a satisfactory section.

Design of Timber Lagging

Assume a 4-in by 14-in (102-mm by 356-mm) Douglas fir-larch grade #1 and better timber plank (figure 17). Allowable design values (NDS 1991 table 4A) are:[16]

F_b' $= F_b\ C_m\ C_D\ C_F\ C_r\ C_{fu}$
$= 1{,}150$ lbf/in^2 x 1.0 x 0.9 x 1.0 x 1.15 x 1.1 = 1,309 lbf/in^2 (9 MPa)

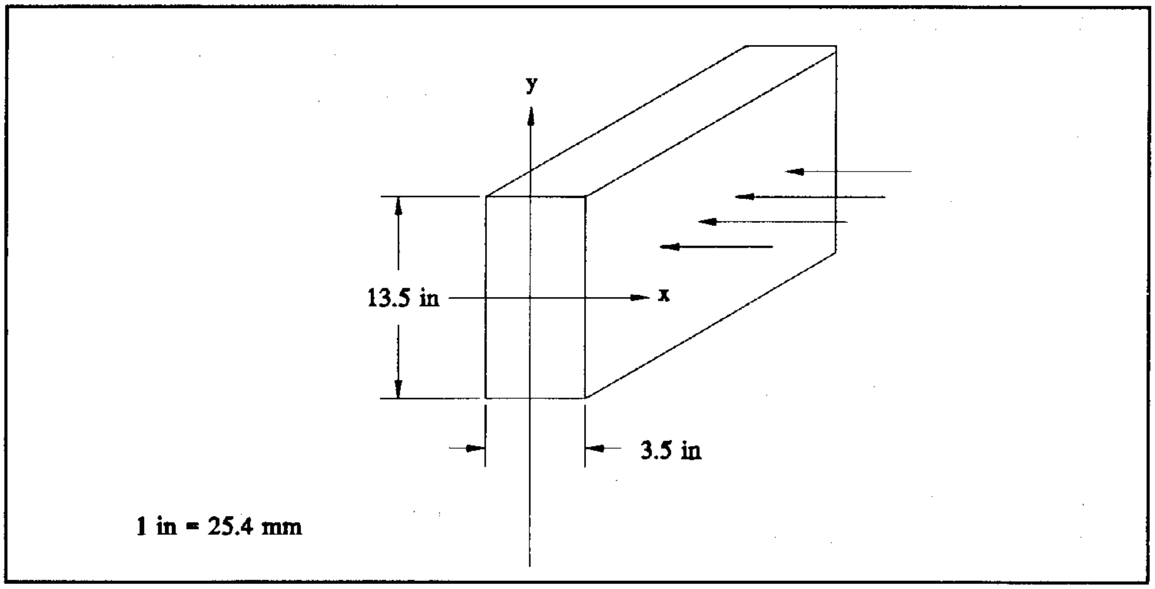

Figure 17. Timber plank.

F_{cp}' $= F_{cp}\ C_m$
$= 625\ \text{lbf/in}^2 \times 0.67 = 419\ \text{lbf/in}^2$ (2.8 MPa)

F_v' $= F_v\ C_m\ C_D = 95\ \text{lbf/in}^2 \times 0.97 \times 0.9 = 83\ \text{lbf/in}^2$ (0.56 MPa)

Section modulus:

$S_y = bd^2/6 = 13.5\ \text{in} \times (3.5\ \text{in})^2/6 = 27.56\ \text{in}^3$ (450 mm^3)

The intensity of the soil pressure on the timber planks in contact with the footing is:

1. Pressure due to soil $= K_a \times \gamma_s \times H$
$= 0.3 \times 100\ \text{lbf/ft}^3 \times 18\ \text{ft} = 540\ \text{lbf/ft}^2$ (25 kN/m^2)

2. Pressure due to surcharge $= 2\ \text{ft} \times \gamma_s \times K_a$
$= 2\ \text{ft} \times 100\ \text{lbf/ft}^3 \times 0.3 = 60\ \text{lbf/ft}^2$ (3 kN/m^2)

Total pressure $= 540\ \text{lbf/ft}^2 + 60\ \text{lbf/ft}^2 = 600\ \text{lbf/ft}^2$ (28.2 kN/m^2)

Uniformly distributed load acting on one plank, $W = 600\ \text{lbf/ft}^2 \times (13.5\ \text{in})/12$ $= 675$ lbf/ft (9.8 kN/m) of length of timber plank.

Bending moment:

$M = WL^2/8$

where,

W = Uniformly distributed load, lbf/ft

L = Spacing of columns minus 3 in (76.2 mm) on each side (see figure 18)
= 5.5 ft (1.67 m)

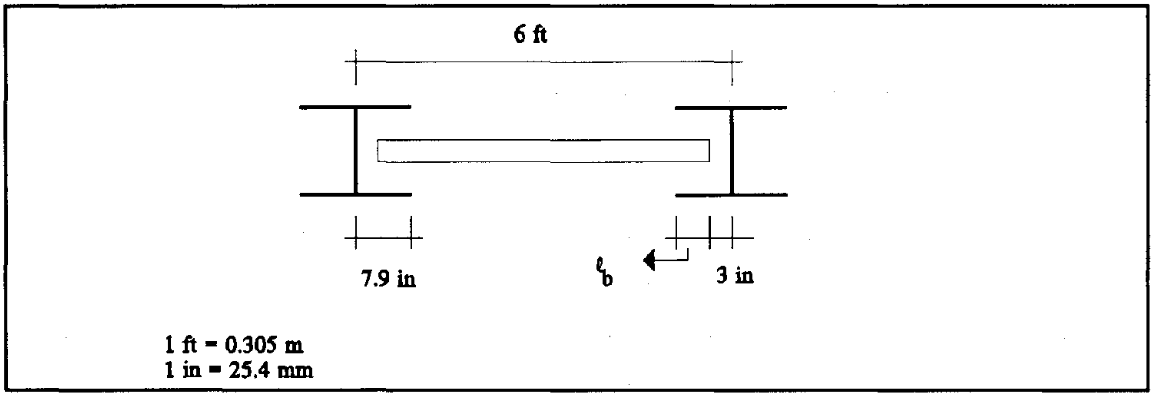

Figure 18. Bearing length for timber planks.

$$M_y = \frac{WL^2}{8} = \frac{675\ lbf/ft \times (5.5\ ft)^2}{8} = 2{,}552\ lbf\text{-}ft = 30{,}628\ lbf\text{-}in\ (3.4\ kN\text{-}m) \quad \textbf{(40)}$$

$$f_b = \frac{M_y}{S_y} = \frac{30{,}628\ lbf\text{-}in}{27.56\ in^3} = 1{,}111\ lbf/in^2\ (7.65\ MPa) \quad \textbf{(41)}$$

$f_b < F'_b$ OK

Bearing length:

The shear force (reaction) at the end of the plank is approximately equal to

$$V = \frac{WL}{2} = \frac{675\ lbf/ft \times 5.5\ ft}{2} = 1{,}856\ lbf\ (8.2\ kN)$$

$$l_b = \frac{V}{F'_{cp} \times d} = \frac{1{,}856\ lbf}{419 \times 13.5\ in} = 0.33\ in\ (8.4\ mm)$$

The supported length is approximately 5.0 in (127 mm), which is OK.

Shear:

$$f_v = 1.5\,\frac{V}{A} = \frac{1.5 \times 1{,}856\,lbf}{13.5\,in \times 3.5\,in} = 59\,lbf/in^2\ (0.4\,MPa)$$

$$f_v < F'_v = 83\ \text{lbf/in}^2\ (0.56\ \text{MPa})$$

Design Summary

The design is summarized in figure 19. For seismic design details and other load conditions, see references 12 and 40 listed at the end of this manual.

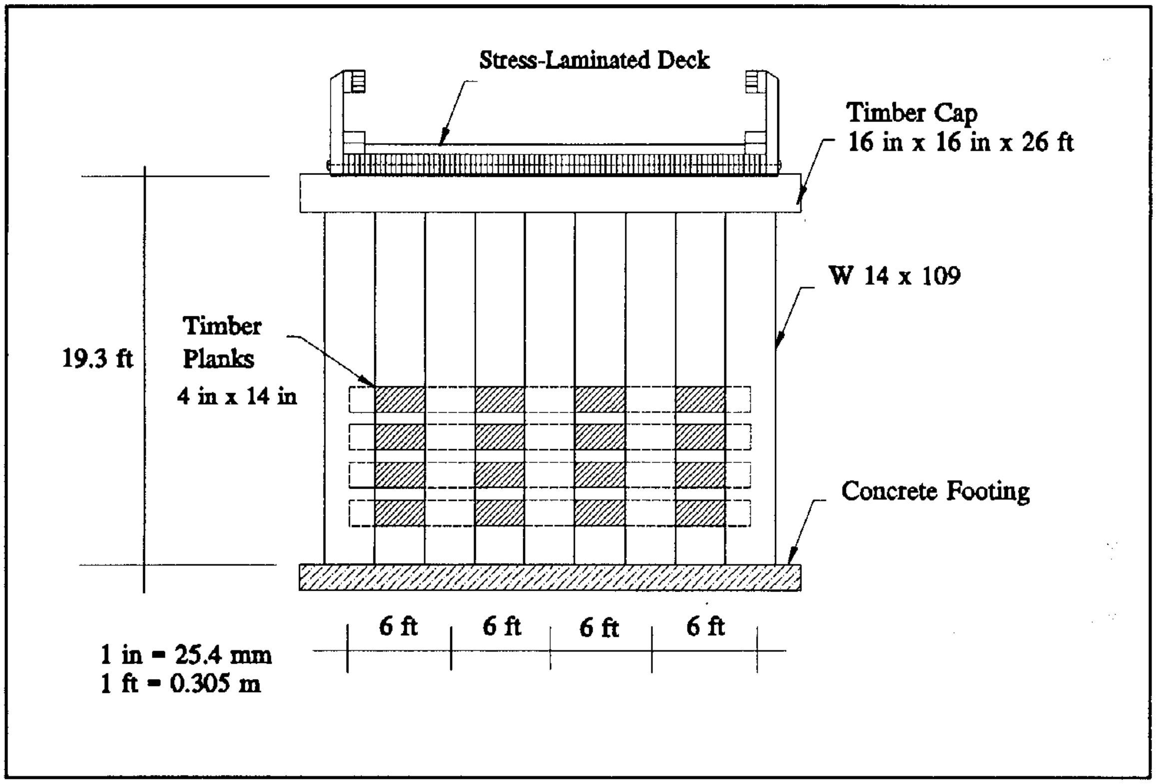

Figure 19. Design summary I.

DESIGN EXAMPLE II: DRIVEN STEEL BENT-PILE ABUTMENT

Assumptions and Design Data

Deck material: Northern Red Oak
Bridge span: 20 ft (6.1 m)
Bridge width: 24 ft (7.32 m)
Loading: HS-20
Lanes: 2
Timber Cap: Douglas fir [Specific weight = 50 lbf/ft^3 (7.85 kN/m^3)]
16 in x 16 in x 26 ft (406 mm x 406 mm x 7.9 m)
Number of columns: 5
Spacing between columns: 6 ft (1.83 m)
Height of abutment: 20 ft (6.1 m)
K_a = Coefficient of active earth pressure: 0.3
γ_s = Unit weight of soil: 100 lbf/ft^3 (15.7 kN/m^3)

Design Calculations

1. Tentatively choose pile size.

Try W 14 x 233

From the *Manual of Steel Construction* (AISC), the section properties are:

d= 16.04 in (407 mm)	S_x = 375 in^3 (6,138 mm^2)	A = 68.5 in^2 (44,182 mm^2)
t_w = 1 in (25.4 mm)	r_x = 6.63 in (168 mm)	r_y = 4.10 in (104 mm)
	I_x = 3,010 in^4 (0.012 m^4)	I_y = 1,150 in^4 (0.0005 m^4)

2. Determine the depth of fixity.

$$D = 1.8 \left[\frac{E I_x}{n_h} \right]^{1/5} \tag{42}$$

where,

n_h = constant of horizontal subgrade reaction, lbf/in^3 (kN/m^3)
= 28 lbf/in^3 (7,588 kN/m^3) for a medium dense soil.[18]

Longitudinal direction

$$D = 1.8 \left[\frac{29 \times 10^6 \times 3{,}010}{28} \right]^{1/5} = 142.5\ in = 12\,ft\ (3.66\,m) \tag{43}$$

Assumed actual embedment length of pile = 40 ft (12.2 m) > 3D (USS Handbook). See figure 20.

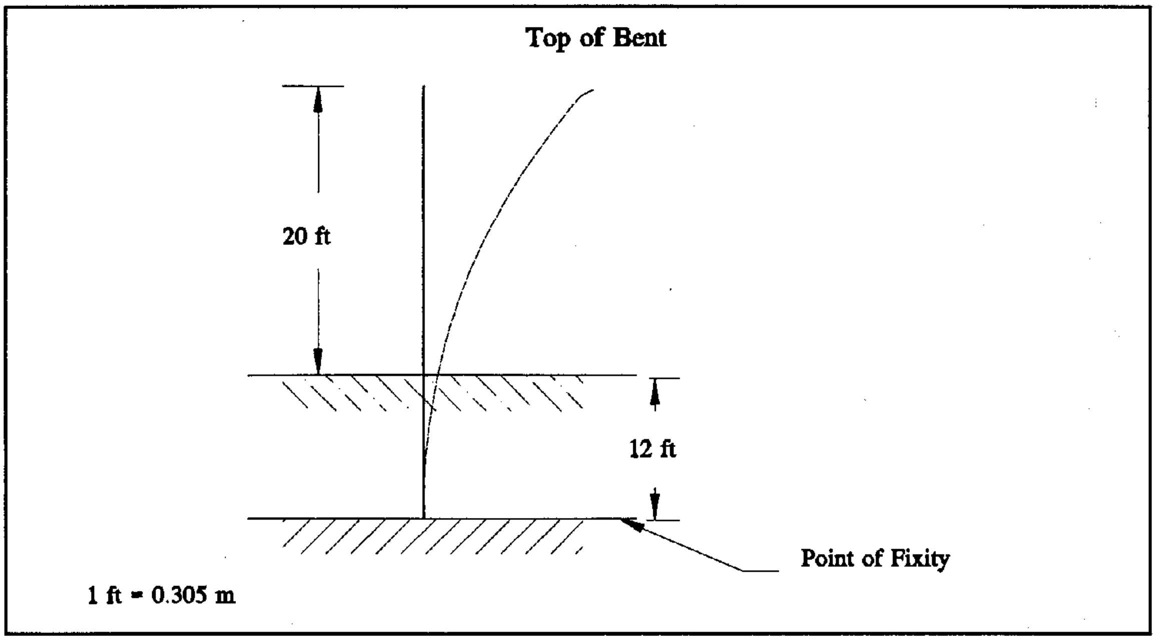

Figure 20. Depth of fixity.

Transverse direction

$$D = 1.8 \left[\frac{29 \times 10^6 \times 1150}{28} \right]^{1/5} = 118\,in = 10\,ft\ (3\,m) \tag{44}$$

3. Determine the allowable compressive stress in the pile based on the appropriate column formula.

Determine the pile's slenderness ratio and allowable compressive stress.

Longitudinal direction

The boundary conditions shown in figure 16 are assumed (conservative), and the maximum moment, $(M_x)_{max}$, is computed at the point of fixity. The effective length factor in the longitudinal direction is $K_x = 2.1$, since the pile is fixed at the bottom and free to rotate and translate at the top.

The pile slenderness ratio for bending about the x-axis (longitudinal direction) is $K_x L_x / r_x$.

Slenderness ratio = 2.1 x (32 ft x 12 in/ft)/ 6.63 in = 121.5

The allowable compressive stress, (AASHTO table 10.32.1A), F_a, when $KL/r \leq C_c = 126.1$ is:

F_a = 16,980 lbf/in^2 - 0.53 $(KL/r_x)^2$
F_a = 16,980 lbf/in^2 - 0.53 $(121.5)^2$ = 9,156 lbf/in^2 (63 MPa)

Transverse direction

In the transverse direction, the piles are considered fixed at the point of fixity and constrained against rotation at the top of the bent with a point of inflection at midheight (see figure 21). Therefore, the maximum moment M_y equals: $(M_y)_{max}$ = 30 ft/2 x (H_x) = 15 ft x H_x (4.5 m x H_x), at the point of fixity and at the top of the pile. The effective length factor in the transverse direction is $K_y = 1.2$.

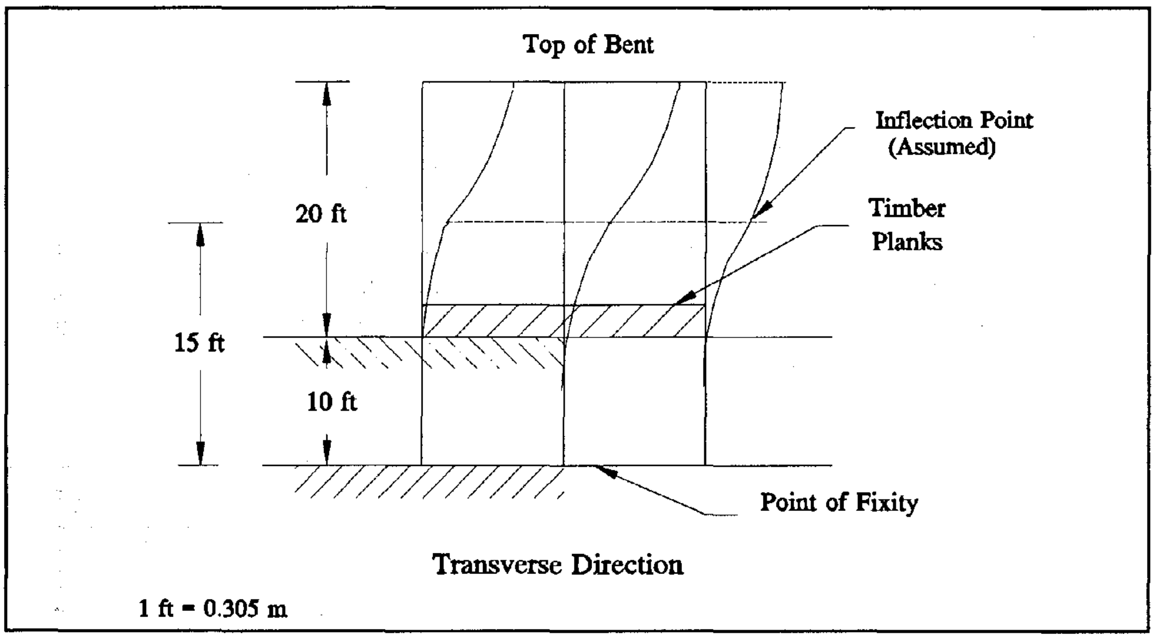

Figure 21. Timber bracing in the transverse direction.

The pile slenderness ratio for bending about the y-axis (transverse direction) is $K_y L_y / r_y$.

Slenderness ratio = 1.2 x (30 ft x 12 in/ft)/ 4.10 in = 105

The allowable compressive stress (AASHTO table 10.32.1A), F_a, when $KL/r \leq C_c = 126.1$ is $F_a = 16{,}980 \text{ lbf/in}^2 - 0.53\,(KL/r_y)^2 = 11{,}136 \text{ lbf/in}^2$ (76 MPa).

4. Determine the vertical and horizontal load per pile.

The total vertical and horizontal loads at the top of the pile bent are determined from AASHTO table 3.22.IA.[12] The AASHTO specifications do not require impact loads for the design of abutments (AASHTO article 3.8.1.2). Considering loadings of types D, L, LF, E, and I, equation 3-10 in AASHTO becomes

$$\text{Group (N)} = \gamma\,[\,\beta_D\,DL + \beta_L\,(L) + \beta_L\,LF + \beta_E\,E + \beta_W\,W + \beta_{WL}\,WL\,]$$

where,

- N = Group number
- γ = Load factor
- DL = Dead load
- L = Live load
- E = Earth pressure
- W = Windload on structure
- WL = Windload on live load
- LF = Longitudinal forces
- β = Coefficients (see AASHTO table 3.22.1A)

The relevant loading combinations are grouped into three categories: I, II, and III. The load factors given in table 1 are multiplied by the corresponding loads, and the products are summed for each load combination, which are subsequently given in tables 2, 3, and 4.

Table 1. Beta coefficients.

Group (N)	DL	L	E	LF	W	WL	% of load
I	1	1	1	0	0	0	100
II	1	0	1	0	1	0	125
III	1	1	1	1	0.3	1	125

Computation of Dead and Live Loads[14]

Total dead load of timber deck: 48,184 lbf (214 kN)

Live load: 41,600 lbf x 2 = 83,200 lbf (370 kN) (AASHTO appendix A)

Total live load: 83,200 lbf (370 kN)

Timber cap weight:

46.2 ft^3 x (50 lbf/ft^3 + 8 lbf/ft^3 "creosote retention") = 2,680 lbf (12 kN)

Total dead load: 48,184 lbf/2 + 2,680 lbf = 26,772 lbf (119 kN)

Total (dead + live) loads on single abutment:

$$26{,}772\ lbf + 83{,}200\ lbf = 109{,}972\ \ lbf\,(489\ kN) \tag{45}$$

Computation of Earth Pressure (see figure 22)

Live load surcharge (AASHTO 3.20.3) = 2 ft (0.6 m)

Uniform load due to surcharge, q = 2 ft x 100 lbf/ft^3 = 200 lbf/ft^2 = 10 kN/m^2

Earth pressure resultant due to surcharge = q x K_a x H
= 200 lbf/ft^2 x 0.3 x 20 ft
= 1,200 lbf/ft
= (18 kN/m) of width of abutment

Earth pressure resultant due to backfill soil = 1/2 x γ_s x H^2 x K_a
= 1/2 x 100 lbf/ft^3 x $(20\ ft)^2$ x 0.3
= 6,000 lbf/ft (88 kN/m)

Total resultant force on abutment, H_y:

1,200 lbf/ft x 24 ft = 28,800 lbf (128 kN)
6,000 lbf/ft x 24 ft = 144,000 lbf (641 kN)

Moment at the point of fixity due to resultant forces (figure 22), M_x:

(28,800 lbf x 22 ft) + (144,000 lbf x 18.67 ft) = 3,322,080 lbf-ft (4.5 kN-m)

Computation of Longitudinal Forces (see figure 22)

AASHTO article 3.9 specifies an approximate longitudinal force based on vehicle loads equal to 5 percent of the live load applied in all lanes carrying traffic in the same direction. The live load used to compute the longitudinal force is the uniform lane load, specified by AASHTO as 640 lbf/ft (9.3 kN/m) plus the concentrated load for moment, which is 18,000 lbf (80 kN) according to AASHTO.

Longitudinal force = (640 lbf/ft x 20 ft + 18,000 lbf) x 0.05 x 2 = 3,080 lbf
= 14 kN

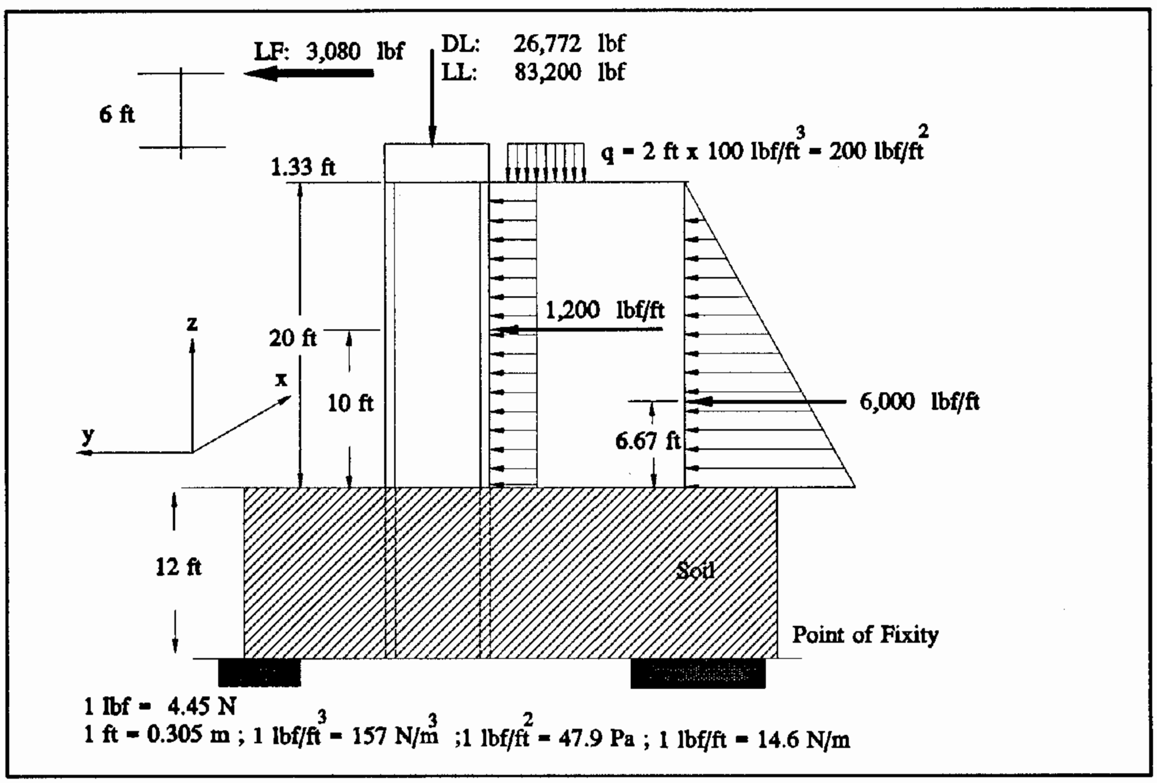

Figure 22. Forces on H-pile.

Computation of Wind Loads (AASHTO article 3.15)

(a) Wind load on superstructure.

Magnitude of wind load acting on superstructure = (area of deck, posts, and rail as seen in the transverse elevation) x (transverse or longitudinal wind loads as specified in AASHTO article 3.15.2.1) (figure 23). The load is applied at the center of gravity of the exposed area.

Area of deck, posts, and rails = 72 ft^2 (7 m^2) (see reference 14)

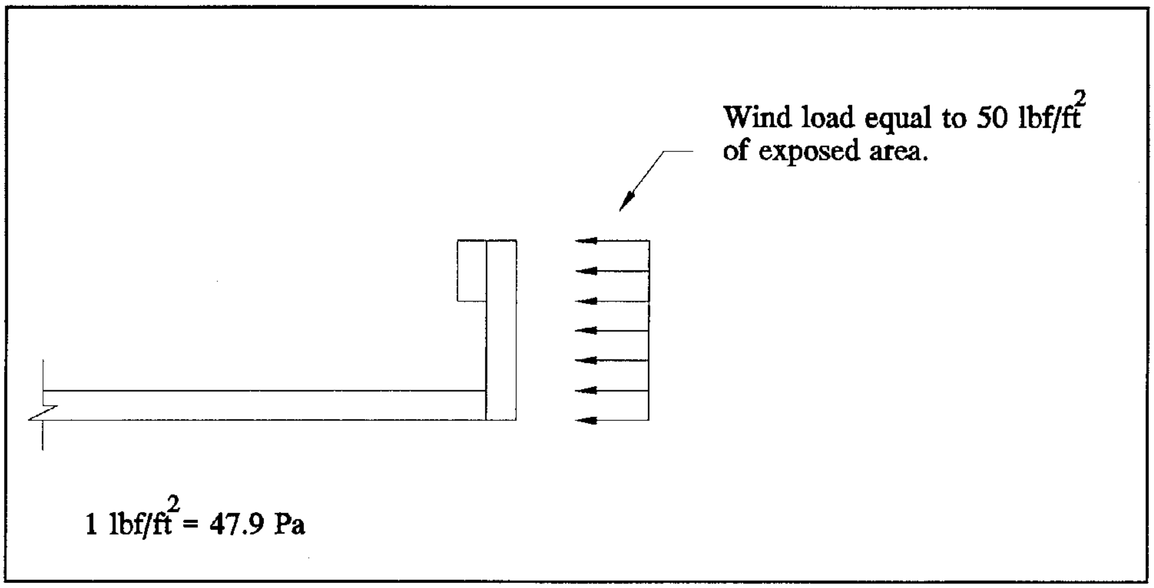

Figure 23. Wind load on structure.

Wind load = 72 ft^2 x 50 lbf/ft^2 = 3,600 lbf (16 kN) (transverse direction)
Wind load = 72 ft^2 x 12 lbf/ft^2 = 864 lbf (3.8 kN) (longitudinal direction)

(b) Wind load on live load.

Magnitude of wind load acting on the live load = (span length) x (unit wind on a moving load as specified in AASHTO 3.15.2.1.2).[12] The wind load is assumed to act 6 ft (0.6 m) above the roadway.

Wind load = 20 ft x 100 lbf/ft = 2,000 lbf (9 kN) (transverse direction)
Wind load = 20 ft x 40 lbf/ft = 800 lbf (3.8 kN) (longitudinal direction)

Total vertical and horizontal loads and moments (M_y) are determined at the point of fixity and given in tables 2, 3, and 4. In addition, overturning moments (M_x) are determined at the point of fixity. Figures 24, 25, and 26 show the loads and their location on the abutment in the x-direction and the y-direction.

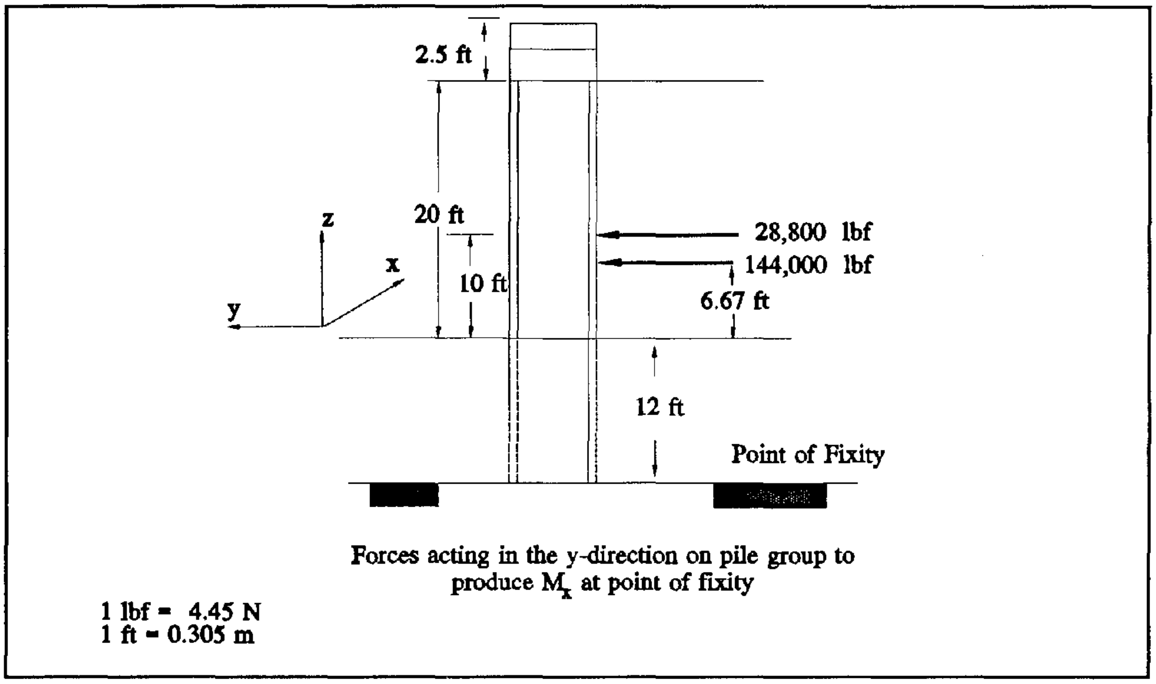

Figure 24. Group I loading.

Table 2. Group I loading.

Loading	Vertical Loads F_v (lbf)	Horizontal Loads Longitudinal (H_y) (lbf)	Moment M_x (lbf-ft)
DL	26,772	-	-
L	83,200	-	-
E	-	28,800 + 144,000	3,322,080
Σ	109,972	172,800	3,322,080

Note: 1 lbf = 4.45 N
1 lbf-ft = 1.35 N-m

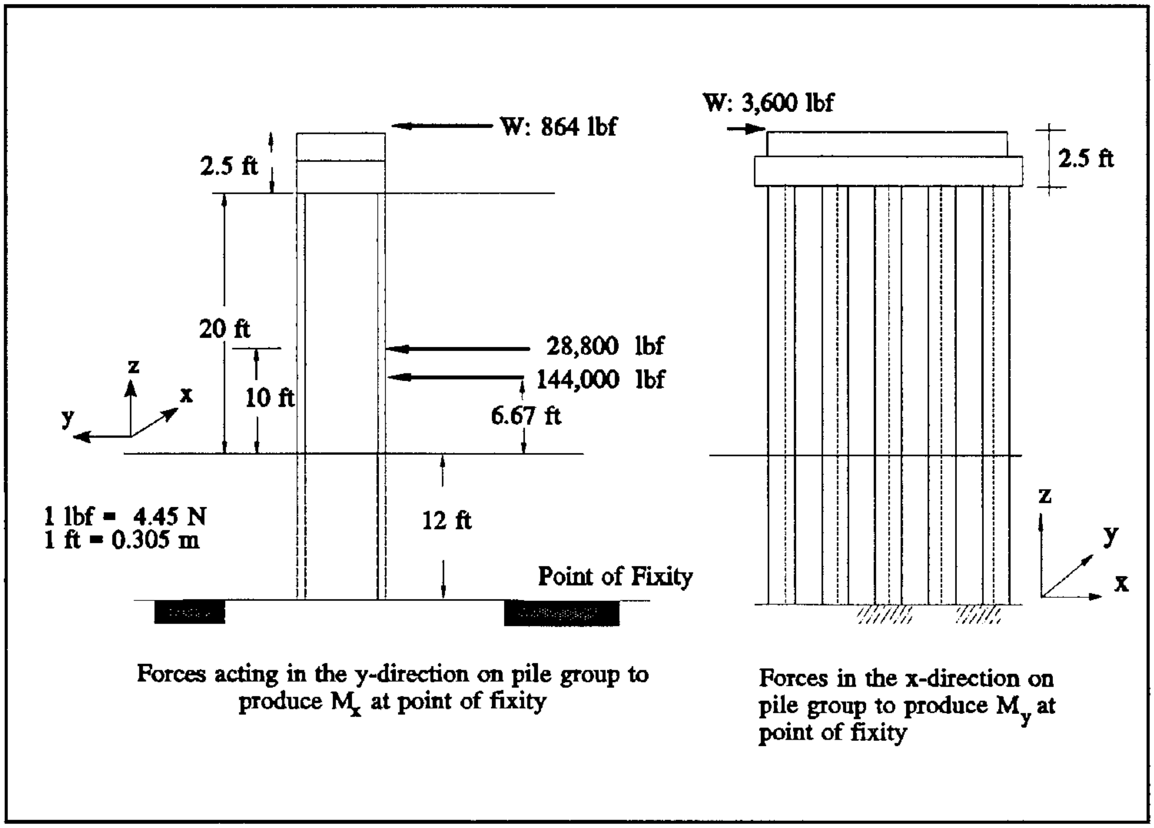

Figure 25. Group II loading.

Table 3. Group II loading.

Loading	F_v (lbf)	Transverse Direction		Longitudinal Direction	
		H_x (lbf)	M_y (lbf-ft)	H_y (lbf)	M_x (lbf-ft)
DL	26,772	-	-	-	-
E	-	-	-	172,800	3,322,080
W (bridge)	-	3,600	54,000	864	29,808
Σ	26,772	3,600	54,000	173,664	3,351,888

Note: 1 lbf = 4.45 N
1 lbf-ft = 1.35 N-m

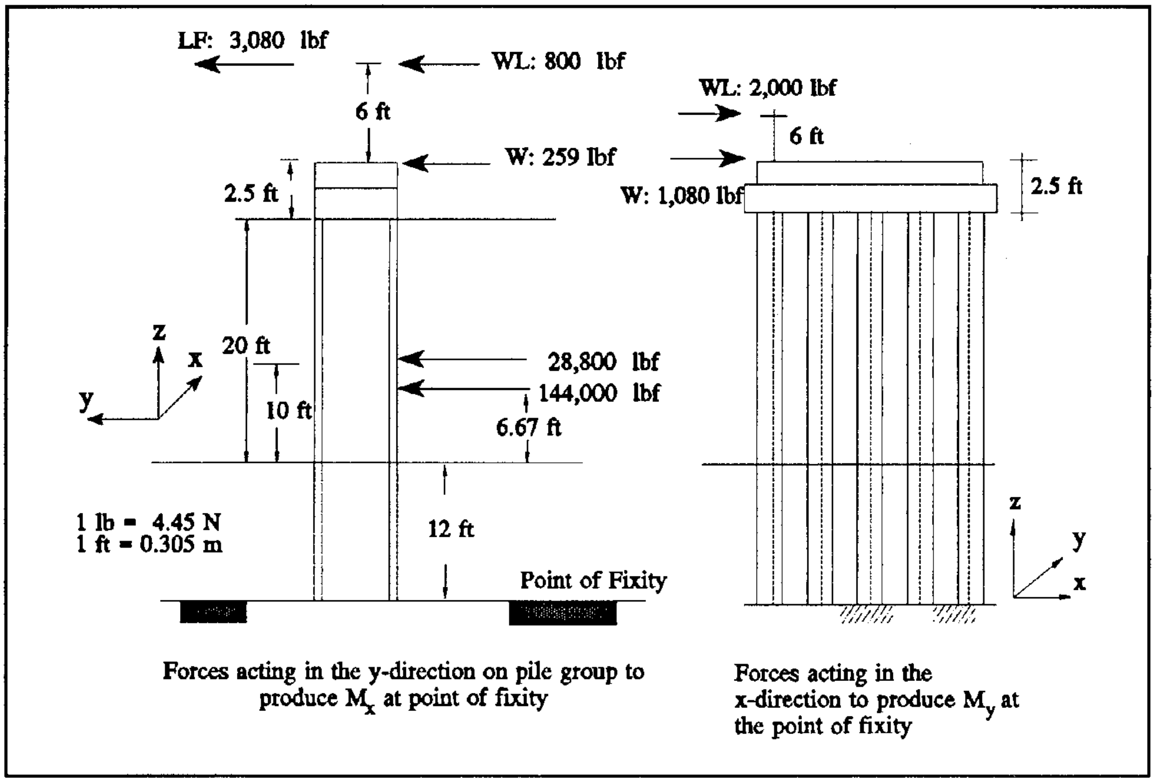

Figure 26. Group III loading.

Table 4. Group III loading.

Loading	F_v (lbf)	Transverse Direction		Longitudinal Direction	
		H_x (lbf)	M_y (lbf-ft)	H_y (lbf)	M_x (lbf-ft)
DL	26,772	-	-	-	-
L	83,200	-	-	-	-
E	-	-	-	172,800	3,322,080
LF	-	-	-	3,080	124,740
W x 0.3	-	1,080	16,200	259	8,935
WL	-	2,000	30,000	800	32,400
Σ	109,972	3,080	46,200	176,930	3,488,155

Note: 1 lbf = 4.45 N
1 lbf-ft = 1.35 N-m

Distribution of loads in pile group

Calculate the pile loads at the top of the pile bent by using (see figure 13):

$$Q_m = \frac{F_v}{n} \pm \frac{M_y x}{\Sigma (x^2)} \tag{46}$$

$\Sigma (x^2) = 2 (0^2 + 6^2 + 12^2) = 360 \text{ ft}^2 (33.4 \text{ m}^2)$

Group I:

$Q_m = 109{,}972$ lbf/5 = 21,944 lbf (98 kN)

Group II:

Max. Q_m = 109,972 lbf/5 + 9,000 lbf-ft (12 ft)/360 ft^2 = 22,294 lbf (99 kN)
Min. Q_m = 109,972 lbf/5 - 9,000 lbf-ft (12 ft)/360 ft^2 = 21,694 lbf (97 kN)

Group III:

Max. Q_m = 109,972 lbf/5 + 19,700 lbf-ft (12 ft)/360 ft^2 = 22,651 lbf (101 kN)
Min. Q_m = 109,972 lbf/5 - 19,700 lbf-ft (12 ft)/360 ft^2 = 21,337 lbf (95 kN)

5. **Determine the maximum stresses in the pile due to axial load plus bending loads.**

According to AASHTO article 10.36, all members subjected to both axial compression and bending stress shall be proportioned to satisfy the following requirement:[12]

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - \frac{f_a}{F'_{ex}}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - \frac{f_a}{F'_{ey}}) F_{by}} \le 1 \tag{47}$$

where,

$$F'_e = \frac{\pi^2 E}{F.S.\ (K_b L_b / r_b)^2} \tag{48}$$

Group I:

F_v = 109,972 lbf (489 kN) M_x = 3,322,080 lbf-ft (4,518 kN-m)
Q_{max} = 21,994 lbf (98 kN)

f_a = 21,994 lbf/68.5 in^2 = 321 lbf/in^2 (2.2 MPa)

$f_{bx} = M_x/S_x$ = (3,322,080 lbf-ft/5) x (12 in/ft)/375 = 21,261 lbf/in^2 (146 MPa)

$$F'_{ex} = \frac{\pi^2 \times 29 \times 10^6}{2.12\ (121.5)^2} = 9{,}130\ lbf/in^2\,(63\,MPa) \tag{49}$$

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \dfrac{f_a}{F'_{ex}}\right) F_{bx}} \leq 1.0 \tag{50}$$

check

$$\frac{321\ lbf/in^2}{9{,}156\ lbf/in^2} + \frac{0.85\ \times\ 21{,}261\ lbf/in^2}{\left(1 - \dfrac{321\ lbf/in^2}{9{,}130\ lbf/in^2}\right) 20{,}000\ lbf/in^2} = 0.04 + 0.94 = 0.98 < 1\ ok \tag{51}$$

Group II:

F_v= 26,772 lbf (119 kN) H_x = 3,600 lbf (16 kN) M_x = 3,351,888 lbf-ft
Q_{max} = 22,294 lbf (99 kN) = (4,558 kN-m)

H_x = 3,600 lbf/5 = 720 lbf (3.2 kN) per pile

M_y = 15 ft (H_x) = 15 ft x 720 lbf = 10,800 lbf-ft (14.68 kN-m) per pile

M_x = 3,351,888 lbf/5 = 670,377 lbf-ft (911 kN-m) per pile

f_a = 22,294 lbf/68.5 in^2= 325 lbf/in^2 (2.2 MPa)

f_{bx} = (670,377 lbf-ft x 12 in/ft)/375 = 21,452 lbf/in^2 (147 MPa)

f_{by} = (10,800 lbf-ft x 12 in/ft)/145 = 893 lbf/in^2 (6 MPa)

F'_{ex} = 9,130 lbf/in^2 (63 MPa)

$$F'_{ey} = \frac{\pi^2 \times 29 \times 10^6}{2.12\ (105)^2} = 12{,}233\ lbf/in^2\ (84\ MPa) \tag{52}$$

$$\left[\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - \frac{f_a}{F'_{ex}})\, F_{bx}} + \frac{C_{my} f_{by}}{(1 - \frac{f_a}{F'_{ey}})\, F_{by}} \right] \frac{1}{1.25} \le 1 \tag{53}$$

$$\left[\frac{325}{9{,}156} + \frac{0.85 \times 21{,}452}{(1 - \frac{325}{9{,}130})\, 20{,}000} + \frac{0.85 \times 893}{(1 - \frac{325}{12{,}233})\, 20{,}000} \right] \frac{1}{1.25} \tag{54}$$

[0.04 + 0.95 + 0.04] x (1/1.25) = 0.82 < 1.0 OK

Group III:

F_v = 109,972 lbf (489 kN) H_x = 3,080 lbf (13.7 kN) M_x = 3,488,155 lbf-ft
Q_{max} = 22,651 lbf (101 kN) = (4,743 kN-m)

H_x = 3,080 lbf/5 = 616 lbf (2.7 kN) per pile

M_y = 15 ft (H_x) = 15 ft x 616 lbf = 9,240 lbf-ft (12.5 kN-m) per pile

M_x = 3,488,155 lbf/5 = 697,631 lbf-ft (948 kN-m) per pile

f_a = 22,651 lbf/68.5 in^2 = 331 lbf/in^2 (2.3 MPa)

f_{bx} = (697,631 lbf-ft x 12 in/ft)/375 = 22,324 lbf/in^2 (153 MPa)

f_{by} = (9,240 lbf-ft x 12 in/ft)/145 = 764 lbf/in^2 (5.2 MPa)

F'_{ex} = 9,130 lbf/in^2 (63 MPa)

F'_{ey} = 12,233 lbf/in^2 (84 MPa)

$$\left[\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{(1 - \frac{f_a}{F'_{ex}}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - \frac{f_a}{F'_{ey}}) F_{by}} \right] \frac{1}{1.25} \leq 1 \tag{55}$$

$$\left[\frac{331}{9,156} + \frac{0.85 \times 22,324}{(1 - \frac{331}{9,130}) 20,000} + \frac{0.85 \times 764}{(1 - \frac{331}{12,233}) 20,000} \right] \frac{1}{1.25} \tag{56}$$

$$[0.04 + 0.98 + 0.03] \times (1/1.25) = 0.84 < 1.0 \quad \text{OK}$$

Therefore, a W 14 x 233 is a satisfactory section. The design of timber lagging follows the procedure given in example I. Timber cap design follows the procedure given in example I for timber piles.

Design Summary

The design is summarized in figure 27. For seismic design and other load conditions, see references 12, 39, and 40 listed at the end of this manual.

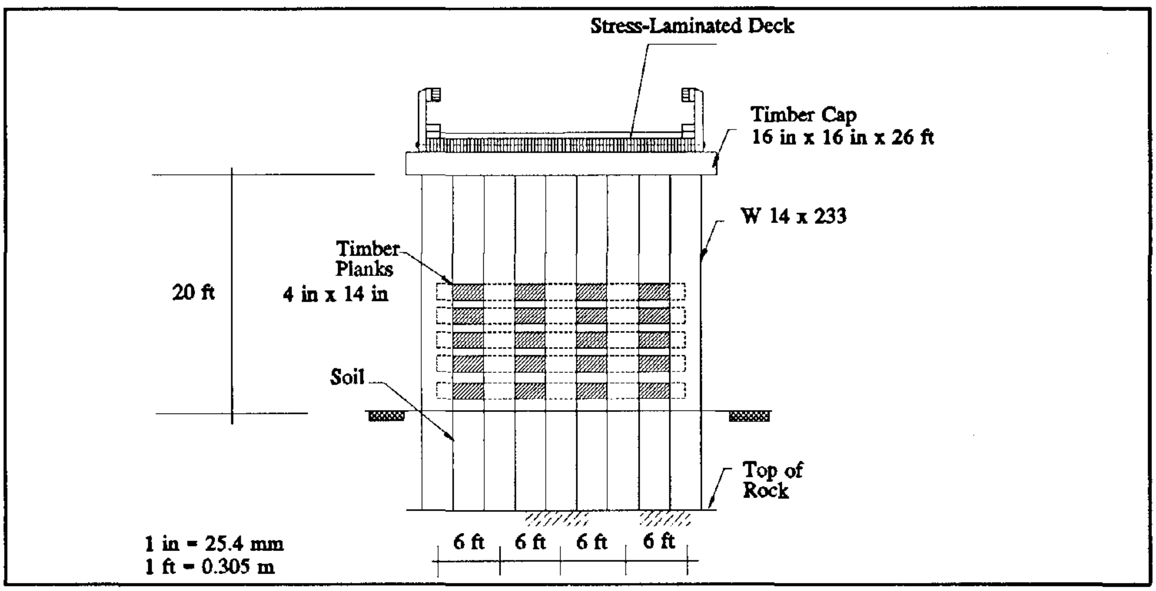

Figure 27. Design summary II.

CHAPTER 4. CULVERT DESIGN

INTRODUCTION

With the development of the interstate highways and the construction of complicated interchanges and wider pavements, the requirements for highway drainage have become increasingly critical.

The term "culvert" applies to all closed conduits used for drainage, and by definition, "structures over 20 ft (6.1 m) in span parallel to the roadway are usually called bridges, and structures less than 20 ft (6.1 m) in span are called culverts, even though they also support traffic loads."(22)

Considerable research in the United States has developed design, construction, and inspection guidelines for reinforced concrete and corrugated steel culverts.(23,24) However, no general procedures exist for the design of timber culverts. The need for approved design procedures for timber culverts is especially significant, because culverts constructed from pressure-treated structural lumber resist freezing and thawing cycles, are noncorrosive and resilient, and can be designed for a variety of site conditions.

The objectives of this chapter are: (1) to present a design approach for one type of commercially available timber culvert based on the AASHTO wheel load distribution, and (2) to recommend guidelines for culvert construction and maintenance procedures.

This chapter presents practical suggestions for the structural design and construction of timber culverts. General considerations for the design of timber culverts is presented first, followed by a design procedure and a design example. In addition, some construction guidelines are given to ensure satisfactory performance of the system.

GENERAL CONSIDERATIONS FOR THE DESIGN OF TIMBER CULVERTS

Culverts are transverse drains under highways, railroads, and other embankments. Most culverts are manufactured in various shapes and sizes from corrugated steel, concrete, or timber.

Economical designs of highway culverts require careful consideration of their hydraulics, location, shape, and construction procedures. To avoid flooding, a culvert must be of a size adequate to carry the discharge from the design storm. Designs of highway culverts also require careful attention to structural details. This is particularly true for culverts of significant length or culverts with substantial fill-heights. The culvert design considerations discussed in this section include hydraulics, culvert location, culvert length, and loads.

Hydraulic Design of Culverts

The aim in hydraulic design is to determine the proper size of a culvert to accommodate the flow of the stream over the expected life-span of the structure. Hydrology, which is the study of the frequency and intensity of rainfall, is used to establish the discharge to be used in culvert design. With the design discharge determined, the size of the culvert is chosen to provide the necessary hydraulic capacity. While much of the material presented in this chapter refers specifically to structural design, it is assumed that proper hydraulic calculations have been performed to determine a specific culvert size. For further guidance on culvert hydraulics see *AASHTO Guide on Hydraulic Design of Culverts and Concrete Culverts and Conduits*.[25]

Culvert Location

Culvert location involves the alignment and grade with respect to the roadway and stream. Culvert location is important to provide the stream with a direct entrance and a direct exit. Any abrupt change in direction at the ends will retard the flow and make necessary a larger structure. Direct inlets and outlets, if not present, can be provided by means of a channel-change or a skewed alignment (figure 28). An ideal grade line for a culvert is one that does not produce silting nor excessive velocities. A slope of 1 to 2 percent is advisable to give a gradient equal to or greater than the critical slope, provided the velocity falls within permissible limits. In general, a minimum slope of 0.5 ft in 100 ft (0.15 m in 30.5 m) will avoid sedimentation. For more information on culvert alignment and grade, see references 24 and 26.

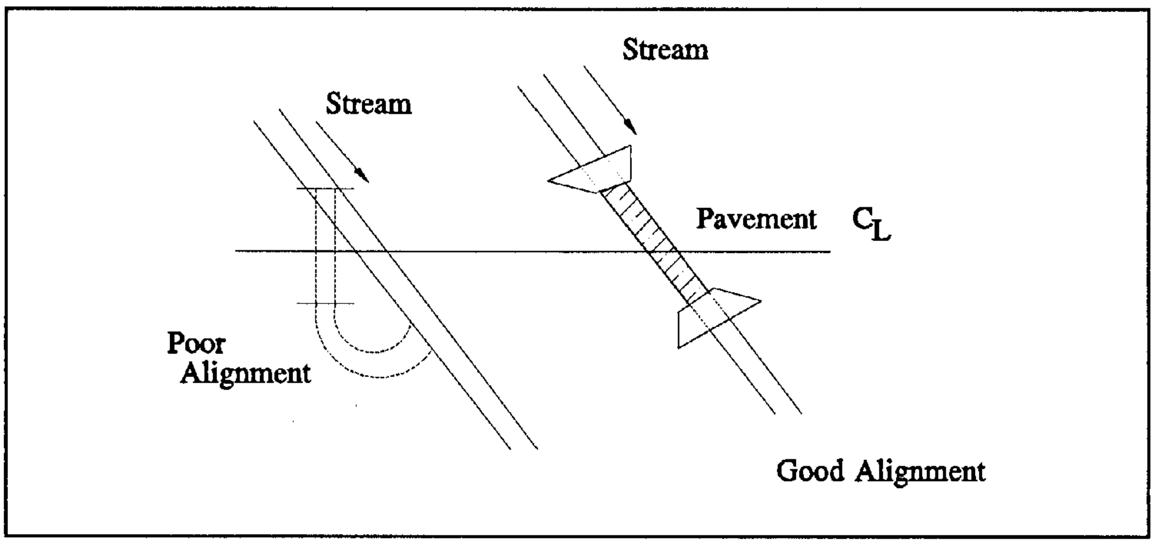

Figure 28. Culvert alignment.

Culvert Length

The required length of culvert depends on the width of the roadway, the height of fill, the slope and skew of culvert, and the characteristics of headwalls and wingwalls. In general, a culvert should be longer than the roadway width to avoid clogging of the ends with sediment. Based on figure 29, the length of culvert can be determined by adding twice the slope ratio times the height of fill to the width of the roadway (R_W).[24]

$$Culvert\,Length = \left[2 \times \left(\frac{h}{v}\right) \times H \right] + R_W \tag{57}$$

Example: A 24-ft (7.32-m) roadway with two to one side slopes and a height of fill equal to 3 ft (0.915 m). The required culvert length is 24 ft + [2(2/1) x 3 ft] = 36 ft (11 m).

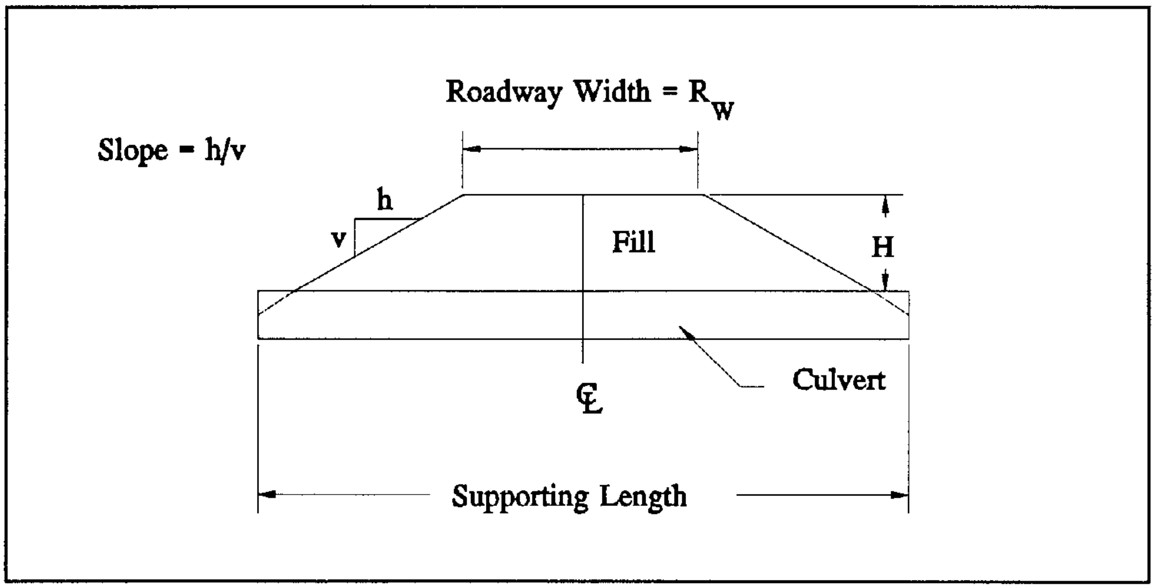

Figure 29. Culvert length.

Loads on Culverts

The design of culverts is treated as a soil-structure interaction problem, and because the magnitude and distribution of earth loads on culverts depend on the relative stiffness of the culvert and soil, current design methods distinguish between a rigid (e.g., reinforced concrete) culvert and a flexible (e.g., corrugated steel or aluminum) culvert. However, presently, there is no established design methodology for timber culverts, which are of intermediate stiffness between rigid and flexible culverts. The procedure presented in this chapter for the design of timber culverts follows the design theory for rigid culverts. The loads acting on the culvert are determined following the Marston-Spangler approach.[27,28]

Culverts must be designed to support dead loads due to earth overburden and live loads due to traffic passing over the culvert. This requires consideration of fill height, type and strength of pavement, and traffic loads. Rigid pavements distribute loads over wide subgrade areas; and therefore, live loads of small intensity are transferred to the buried structure. However, significant live loads are transmitted through flexible pavements. In addition, live load distribution diminishes rapidly as the depth of cover over the culvert increases, and traffic loads may be disregarded when the fill exceeds 8 ft (2.4 m) (AASHTO article 6.4).[12]

Dead Loads: Dead loads include the earth load or weight of the soil over the culvert and any added surcharge loads. In general, 120 lbf/ft^3 (19 kN/m^3) for the actual weight of soil is assumed.

Live Loads (AASHTO Wheel Load Distribution; see AASHTO article 6.4): The transmission and distribution of wheel loads through soil is generally assumed to follow the principles of elasticity. When the depth of fill is less than 2 ft (0.6 m), AASHTO specifies that the wheel load be distributed as on an exposed slab (i.e., a bridge deck).

When the depth of fill is 2 ft (0.6 m) or more, concentrated applied wheel loads are distributed over a 20-in by 10-in (508-mm by 254-mm) patch area, which is distributed over a rectangular area of dimensions 1.75 x H larger than the wheel patch area (figure 30).

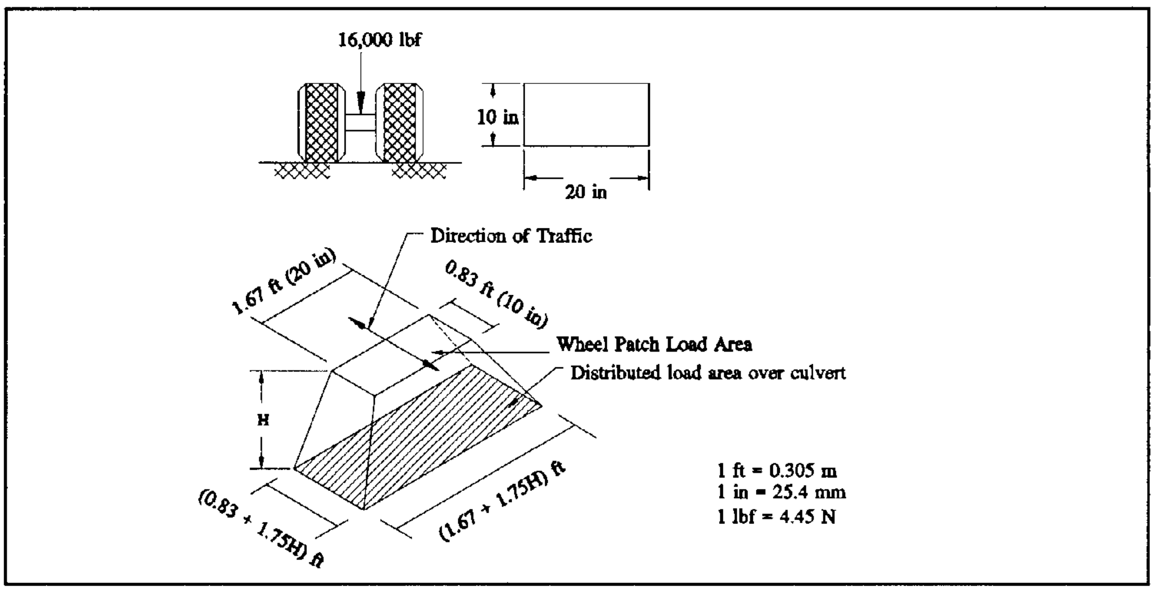

Figure 30. Surface contact area for single wheel load.

When areas from several concentrations overlap, the total load is considered uniformly distributed over the area defined by the outside limits of the individual areas. The total width of distribution should not exceed the total width of the supporting slab. For single-span culverts, the effect of live load may be neglected when the fill is more than 8 ft (2.4 m) and exceeds the span length. For multiple spans, the live load may be neglected when the depth of fill exceeds the distance between the end supports or abutments (AASHTO article 6.4).

Lateral Earth Pressures: Active lateral earth pressures act on the culvert walls and are a direct function of the vertical pressure. The equation for computing lateral earth pressure according to the Rankine theory is:[10]

$$P_a = \frac{1}{2} \gamma_s H^2 K_a \tag{58}$$

where,

P_a = Active earth pressure
γ_s = Unit weight of backfill soil
H = Height of embankment
K_a = Coefficient of active earth pressure ≈ 0.3

DESIGN PROCEDURES

Design procedures for culverts of either reinforced concrete or corrugated metal are available that treat the first one as a rigid structure and the second one as a flexible structure. In this section we present a design procedure for a specific type of timber culvert that is being used currently for secondary roads. The details of this timber box culvert are given in figure 31. The system consists of mechanically laminated lumber, typically rough-sawn 3-in by 7-in to 8-in (76.2-mm by 178-mm to 203-mm) members using galvanized steel dowels to construct modular sections. To allow for end-bearing between a vertical and a horizontal member, one end of every member is notched (see figure 32).

The horizontal and vertical lumber components are assumed to be simply-supported, with a span extending from center-of-notch to center-of-notch. The reaction forces computed for the vertical member are reversed and applied to the horizontal member, which is designed as a beam column (figure 32). Similarly, the vertical member is also designed as a beam-column. The eccentric axial load applied at the center of the notched section bends the member in the opposite direction to that of the applied bending loads, and therefore, the eccentricity of the axial load is neglected, and the axial load is applied at the centroidal axis of the member. The design guidelines for this type of culvert are presented next.

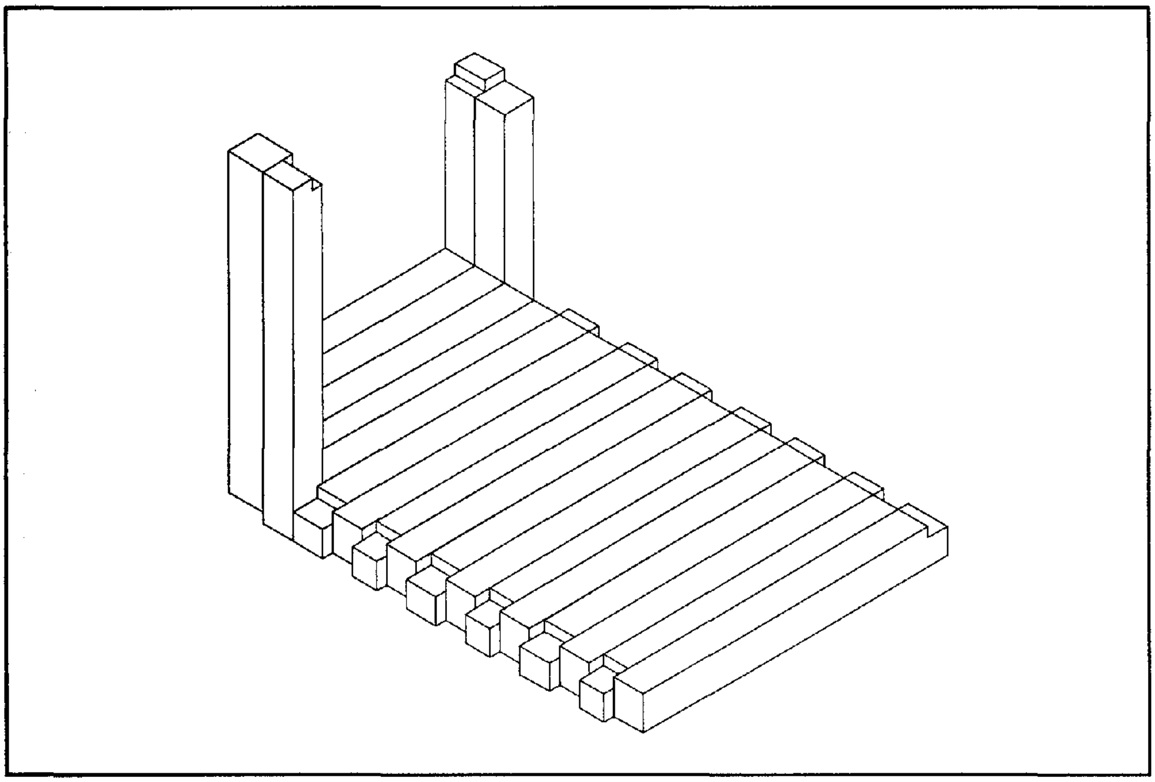

Figure 31. Timber box culvert.

1. **Define culvert location, length, and waterway opening. Recommendations on culvert location, length, and waterway opening are given in the AASHTO guide on hydraulic design of culverts.[25]**

2. **Select a species and grade of lumber for the culvert and determine the allowable design values given in NDS.[16]**

Timber culverts are usually constructed from lumber in the *Joists and Planks* size classification [2 to 4 in (51 to 102 mm) thick, 5 in (127 mm) and wider], and the values for multiple-member are used. Select a species and grade of lumber from NDS, table 4A. Generally, grade No. 1 material is selected, and the following allowable design values are computed:

Bending strength:	$F_b' = F_b\, C_m\, C_D\, C_F\, C_r$
Elastic modulus:	$E_L' = E_L\, C_m$
Compression perpendicular to grain:	$F_{cp}' = F_{cp}\, C_m$
Compression parallel to grain:	$F_c' = F_c\, C_m\, C_D\, C_F\, C_r$
Shear strength:	$F_v' = F_v\, C_m\, C_D$
Bearing parallel to grain:	$F_g' = F_g\, C_D$

where,

C_m	=	Moisture content factor, which has a distinct value *for each design property* given in NDS
C_D	=	Load duration factor from NDS table 2.3.2
C_r	=	Repetitive member factor from NDS table 4A
C_F	=	Size factor from NDS table 4A

3. Compute loads acting on culvert cross section.

In general, culverts are subjected to the following loads:

I. A vertical load uniformly applied over the top of the culvert, which may include both live load and the weight of the earth fill.

Vertical load due to live load: AASHTO wheel load distribution.
Vertical load due to earth fill: Load due to embankment fill may be assumed to be uniformly distributed over the top of the culvert.

II. Active lateral earth pressure symmetrically applied to both sides. This includes a uniform lateral load equal to the pressure intensity at the top of the culvert, and a triangular lateral load representing the increase in active lateral earth pressure over the vertical height of the culvert.

III. Pressure from the contained water when the culvert is full, based on a unit weight of water of 62.5 lbf/ft^3 (9.81 kN/m^3).

The culvert must be designed for the following load combinations: (1) loads I and II, and (2) loads I, II, and III, since the culvert may sometimes be empty and sometimes full of water. Detailed explanation and calculations of these loads are given in a design example.

4. Check beam-column action.

A timber box or rectangular culvert is assumed to be hinged at the corners, and both the horizontal and vertical members are subjected to combined bending and compression axial loads. For the type of culvert shown in figures 31 and 32, the mechanical lamination effect prevents the members from buckling about the weak-axis (both column buckling and beam lateral buckling), and therefore, the horizontal and vertical members are designed for a concentric axial load and bending loads about the strong axis. For this case, the interaction equation given in NDS article 3.9.2 is reduced to:

$$\left[\frac{f_c}{F_c'}\right]^2 + \frac{f_{b1}}{F_{b1}'\left[1-\left(\frac{f_c}{F_{cE1}}\right)\right]} \leq 1.0 \tag{59}$$

where,

f_c = The actual compression stress parallel to grain
$f_c < F_{cE1}$ = $(K_{cE}\ E')/(l_{e1}/d_1)^2$ for uniaxial or biaxial bending

f_{b1} = Actual edgewise bending stress (bending load applied to the narrow face of member, see figure 32)
d_1 = Wide face dimension (shown as d in figure 32)
l_{e1} = Effective column length = $K_e l$
K_e = Buckling length coefficient (NDS appendix G)
K_{cE} = Euler buckling coefficient for columns = 0.3 for visually graded lumber and machine-evaluated lumber
F_{b1}' = Allowable edgewise bending design value

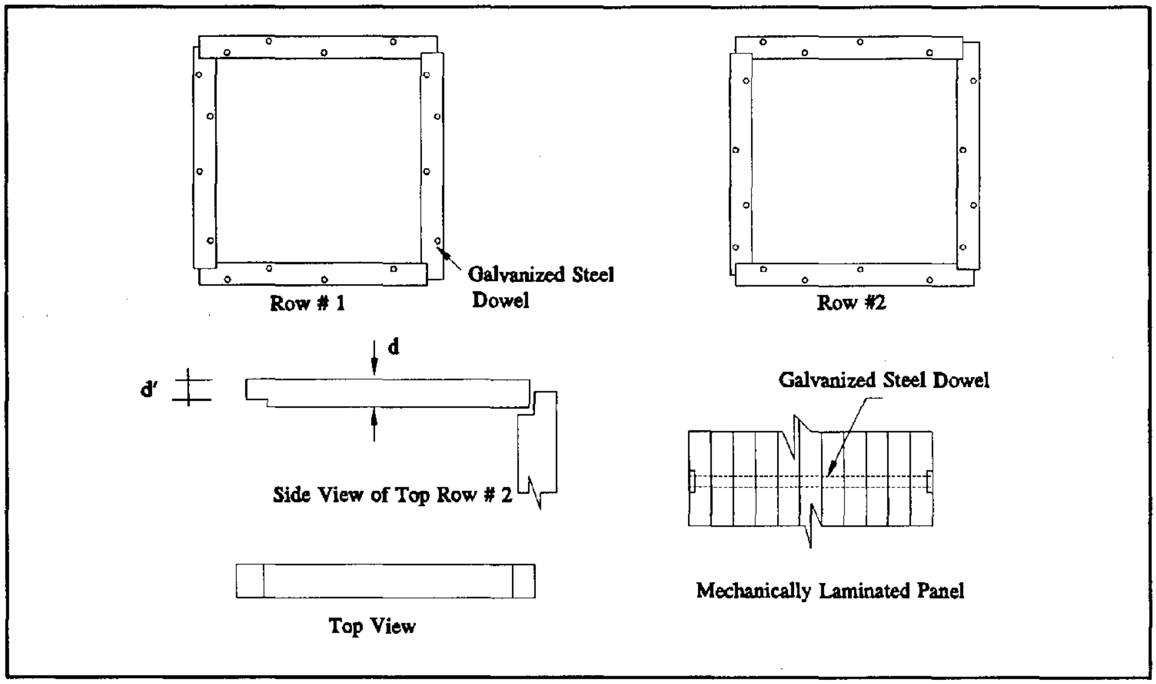

Figure 32. End notches.

5. Check shear at end notches.

In general, notching a beam on the tension side (figure 32) is not recommended, since notches induce tensile stresses normal to the grain, which in combination with horizontal shear tend to cause splitting. The depth of an end-notch on the tension side must not exceed one-fourth the beam depth (NDS 3.2.3.2), and the horizontal shear stress at the notched section is computed from (NDS 3.4.4.1):[16]

$$f_v = \frac{3}{2}\,\frac{V}{bd'}\left[\frac{d}{d'}\right] \tag{60}$$

where,

- f_v = Actual shear stress parallel to grain, lbf/in^2 (kPa)
- V = Maximum shear force at the notch, lbf (N)
- b = Width of the beam, in (mm)
- d = Depth of the beam, in (mm); see figure 32
- d′ = Depth of the beam minus the depth of the notch, in (mm); see figure 32

6. Check bearing stress.

A bearing failure is not a collapse, it is merely a crushing of fibers. To prevent this crushing, the reaction at the support is divided by the bearing area, and the actual bearing stress should not exceed the bearing design value.

Bearing parallel to grain: The actual bearing on the end grain of the member is computed as

$$f_g = \frac{P}{A} \tag{61}$$

where,

- f_g = Actual bearing stress parallel to grain, lbf/in^2 (kPa)
- P = Total applied axial load, lbf (N)
- A = Net bearing area, in^2 (mm^2)

f_g should not exceed F_g'. For requirements of bearing on metal plates or other materials of adequate strength, see NDS 3.10.1.3.

Bearing perpendicular to grain: The actual bearing stress perpendicular to grain is computed as

$$f_{cp} = \frac{R}{A} \tag{62}$$

where,

f_{cp} = Actual stress in compression perpendicular to grain, lbf/in^2 (kPa)
R = Reaction or bearing force at the support, lbf (N)
A = Net bearing area, in^2 (mm^2)

The actual compression stress perpendicular to grain f_{cp} must not exceed the allowable compression F_{cp}' design value computed in step #2 (NDS 3.10.2).

7. Check deflection.

AASHTO specifications do not provide deflection criteria for timber culverts, and the selection of an appropriate deflection limit is a matter of engineering judgment. The acceptable deflection for a member will depend on specific use requirements. Bridge deflection limits are expressed as a fraction of the span, and for culverts, deflection limits can also be expressed as a fraction of the span, which is the length of the culvert parallel to traffic. As a check, the deflection for a simply supported beam with a uniform load can be computed from:

$$\Delta = \frac{5WL^4}{384E'I} \tag{63}$$

where,

W = Magnitude of uniform load, lbf/in (N/m)
L = Span length, in (m)
E' = Modulus of elasticity from step #2, lbf/in^2 (kPa)
I = Moment of inertia about the axis of bending, in^4 (mm^4)

Since the timber culvert design presented in this chapter is assumed to follow the design of a "rigid culvert," a deflection limit of L/800 is used to justify the method presented herein. This deflection limit is recommended by AASHTO for highway bridges.[12]

DESIGN EXAMPLE

It is assumed that a hydraulic design calculation has been performed and a waterway opening of 35 ft^2 (3.25 m^2) is required. A square timber box culvert with a clear opening of 6 ft by 6 ft (1.83 m by 1.83 m) is adequate. Assume that 3-in by 7-in (76.2-mm by 178-mm) structural lumber is used.

1. **Define culvert location, length, and waterway opening.**

Waterway opening:	35 ft^2 (3.25 m^2)
Fill on top of culvert:	3 ft (0.915 m)
Unit weight of fill:	120 lbf/ft^3 (18.84 kN/m^3)
Weight of wearing surface:	150 lbf/ft^3 (23.5 kN/m^3)
Live load:	HS-20
Culvert span:	6 ft (1.83 m)
Roadway width:	24 ft (7.32 m)

Culvert length [assuming a fill slope ratio of 2 (horizontal) to 1 (vertical)] is

$$\textit{Culvert length} = 24\,ft + \left[2 \times \frac{2}{1} \times 3\,ft\right] = 36\,ft\;(11m) \tag{64}$$

2. **Select species and grade of lumber, and determine allowable values.**

Material: Douglas fir-larch grade No. 1 and better (NDS table 4A)

$$F_b' = F_b\, C_m\, C_D\, C_F\, C_r$$
$$= 1{,}150\ lbf/in^2 \times 0.85 \times 0.9 \times 1.3 \times 1.15 = 1{,}315.2\ lbf/in^2\ (9\ MPa)$$

$$E_L' = E_L\, C_m$$
$$= 1.8 \times 10^6 \times 0.9 = 1.62 \times 10^6\ lbf/in^2\ (11{,}162\ MPa)$$

$$F_{cp}' = F_{cp}\, C_m$$
$$= 625\ lbf/in^2 \times 0.67 = 418.75\ lbf/in^2\ (2.88\ MPa)$$

$$F_c' = F_c\, C_m\, C_D\, C_F$$
$$= 1{,}500\ lbf/in^2 \times 0.8 \times 0.9 \times 1.1 = 1{,}188\ lbf/in^2\ (8.18\ MPa)$$

$$F_v' = F_v\, C_m\, C_D$$
$$= 95\ lbf/in^2 \times 0.97 \times 0.9 = 83\ lbf/in^2\ (0.57\ MPa)$$

$$F_g' = F_g\, C_D \text{ (see NDS table 2A)}$$
$$= 2{,}020\ lbf/in^2 \times 0.9 = 1{,}818\ lbf/in^2\ (12.5\ MPa)$$

3. **Compute loads acting on culvert cross section.**

I. *Vertical load due to live load*: The recommended AASHTO wheel load distribution is used, and an AASHTO HS-20 symmetrical load is considered.

Live load distribution for each wheel load (figures 30 and 33):

Parallel-to-traffic direction: 0.83 ft + 1.75 (3 ft) = 6.08 ft (1.85 m)
Normal-to-traffic direction: 1.67 ft + 1.75 (3 ft) = 6.92 ft (2.11 m)

Since the transverse load distribution overlaps at the fill-depth of 3 ft (0.915 m), a uniform live load distributed over the effective width of 22.92 ft (7 m) [i.e., 16 ft + 6.92 ft = 22.95 ft (7 m)] is used (see figure 33), which is computed as:

$$W_{LL} = \frac{16{,}000\,lbf \times 4}{22.92\,ft \times 6.08\,ft} = 459\ lbf/ft^2\ (22\ kN/m^2)$$

For a 1-ft (0.305 m) section, W_{LL} = 459 lbf/ft^2 x 1 ft = 459 lbf/ft
= 6.7 kN/m

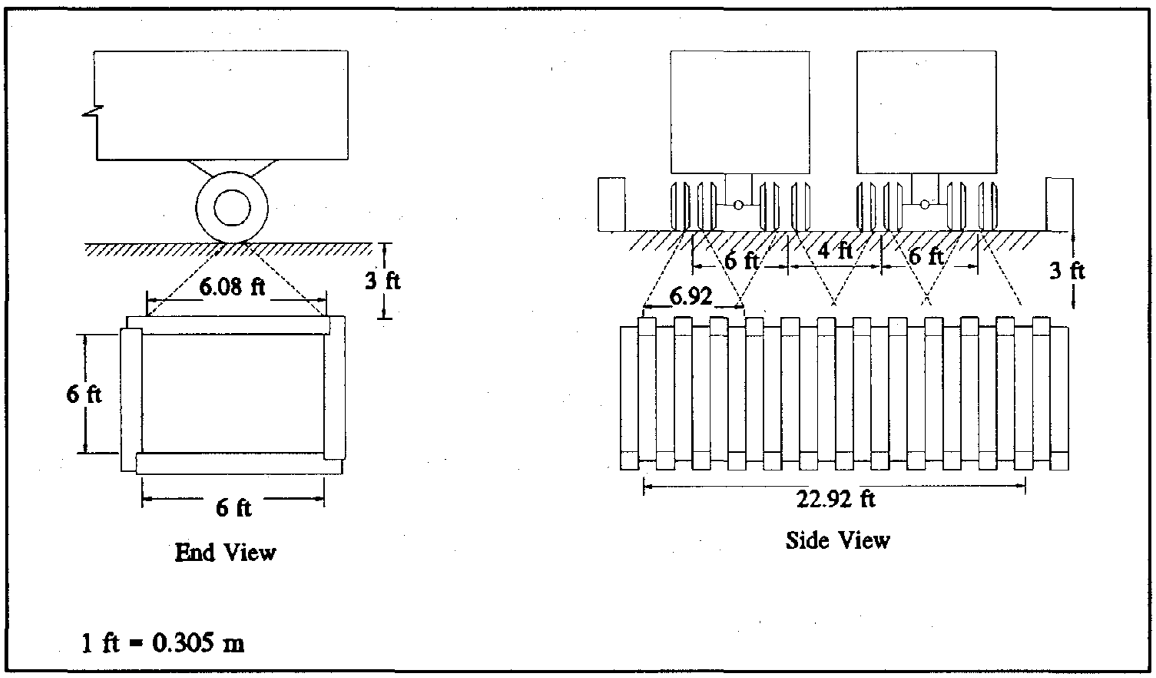

Figure 33. Live load distribution due to HS-20 loading.

Vertical load due to earth fill: Load due to embankment fill may be assumed to be uniformly distributed over the top of the culvert.

Uniformly distributed dead load:

Due to fill: 3 ft x 120 lbf/ft^3 = 360 lbf/ft^2 (17.24 kN/m^2)
Due to 2-in (50.8 mm) wearing surface: 2 in /12 x 150 lbf/ft^3
= 25 lbf/ft^2 (1.19 kN/m^2)

Total distributed dead load, W_{DL} = 360 lbf/ft^2 + 25 lbf/ft^2
= 385 lbf/ft^2
= 18.4 kN/m^2

For a 1-ft (0.305-m) section, W_{DL} = 385 lbf/ft^2 x 1 ft = 385 lbf/ft
= 5.62 kN/m

Total vertical load for a 1-ft (0.305-m) section, $W_{TOTAL} = W_{LL} + W_{DL}$
W_{TOTAL} = 459 lbf/ft + 385 lbf/ft = 844 lbf/ft (12.32 kN/m)

II. Active lateral earth pressure symmetrically applied to both sides. This consists of a uniform lateral load equal to the pressure intensity at the top of the culvert, and a triangular lateral load representing the increase in active lateral earth pressure over the vertical height of the culvert.

Uniform lateral load

Intensity of pressure on top of culvert due to 3-ft (0.915-m) earth fill (figure 34):

q = 3 ft x 120 lbf/ft^3 = 360 lbf/ft^2 (17.24 kN/m^2)

Resultant force, $P_1 = q \times K_a \times H$

P_1 = 360 lbf/ft^2 x 0.3 x 6 ft = 648 lbf/ft (9.46 kN/m)

For a 1-ft (0.305-m) section, P_1 = 648 lbf/ft x 1 ft = 648 lbf (2.88 kN)

Triangular load

Resultant force due to triangular load (figure 35), $P_2 = 1/2\ \gamma\ K_a\ H^2$
P_2 = 1/2 x 120 lbf/ft^3 x 0.3 x $(6\ ft)^2$ = 648 lbf/ft (9.46 kN/m)
For a 1-ft (0.305-m) section, P_2 = 648 lbf/ft x 1 ft = 648 lbf (2.88 kN)

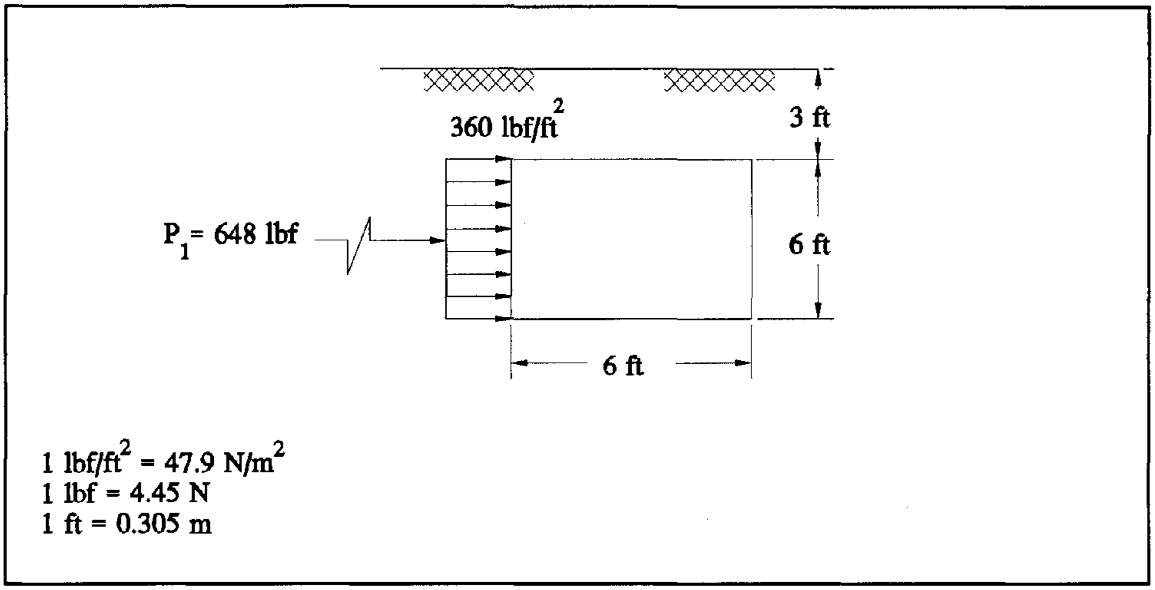

Figure 34. Uniform load due to earth fill.

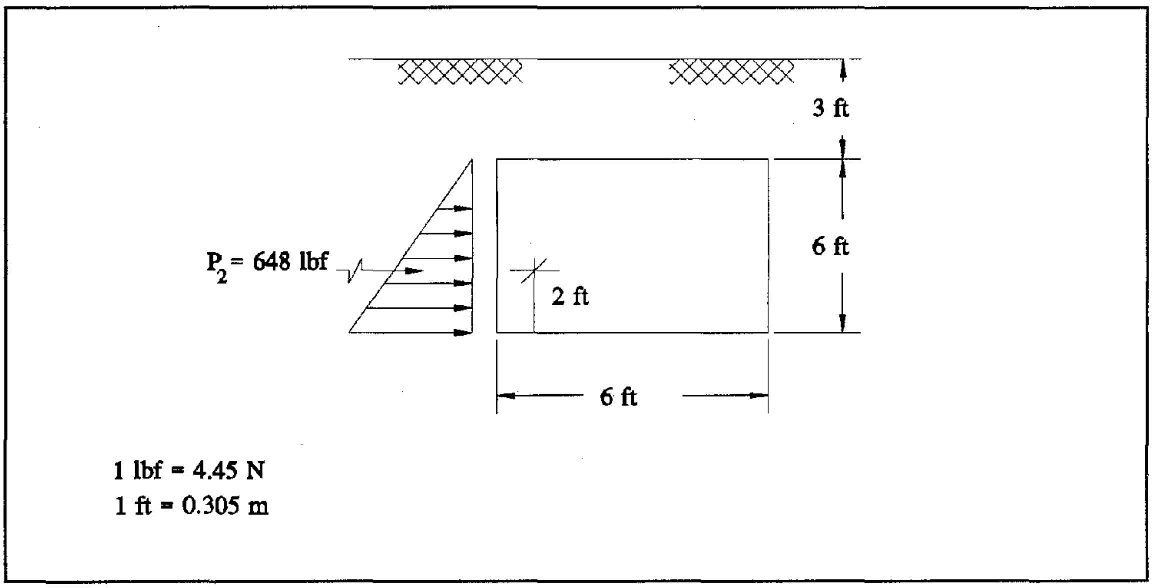

Figure 35. Triangular load.

Uniform lateral load due to live load surcharge (AASHTO 3.20)

Intensity of pressure on top of culvert due to 2-ft (0.61-m) surcharge (figure 36):

$q = 2 \text{ ft} \times 120 \text{ lbf/ft}^3 = 240 \text{ lbf/ft}^2 \ (11.5 \text{ kN/m}^2)$

Resultant force, $P_3 = q \times K_a \times H$

$P_3 = 240 \text{ lbf/ft}^2 \times 0.3 \times 6 \text{ ft} = 432 \text{ lbf/ft (6.3 kN/m)}$

For a 1-ft (0.305-m) section, $P_1 = 432 \text{ lbf/ft} \times 1 \text{ ft} = 432 \text{ lbf (1.92 kN)}$

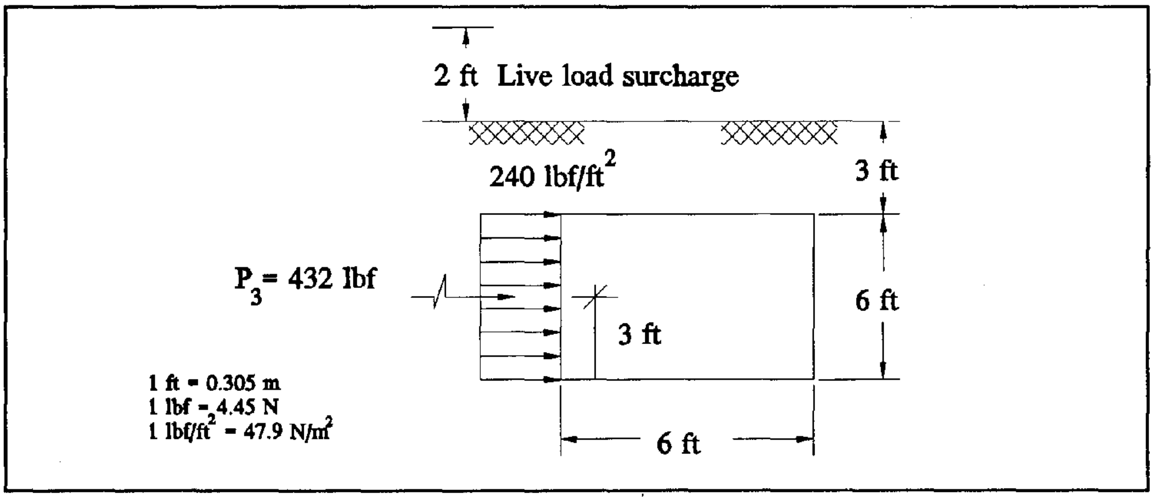

Figure 36. Uniform load due to surcharge.

III. Pressure from the contained water when the culvert is full, based on a unit weight of water of 62.5 lbf/ft³ (9.81 kN/m³).

A triangular pressure distribution is assumed (figure 37), and the resultant force is

$P_w = 1/2\ \gamma_w H^2.$

$P_w = 1/2 \times 62.4 \text{ lbf/ft}^3 \times (6 \text{ ft})^2 = 1{,}123 \text{ lbf/ft (16.4 kN/m)}$

For a 1-ft (0.305-m) section, $P_w = 1{,}123 \text{ lbf/ft} \times 1 \text{ ft} = 1{,}123 \text{ lbf} = 5 \text{ kN}$

By inspection, the most critical load condition is when the culvert is empty, and therefore, a load combination including water will not be considered in this design example.

4. Check beam-column action.

The total loads acting on a 1-ft (0.305-m) culvert cross section due to loads I and II (step #3) are shown in figure 38. Checking for beam-column action in the section requires the determination of the axial load, i.e., the reaction force on the vertical member.

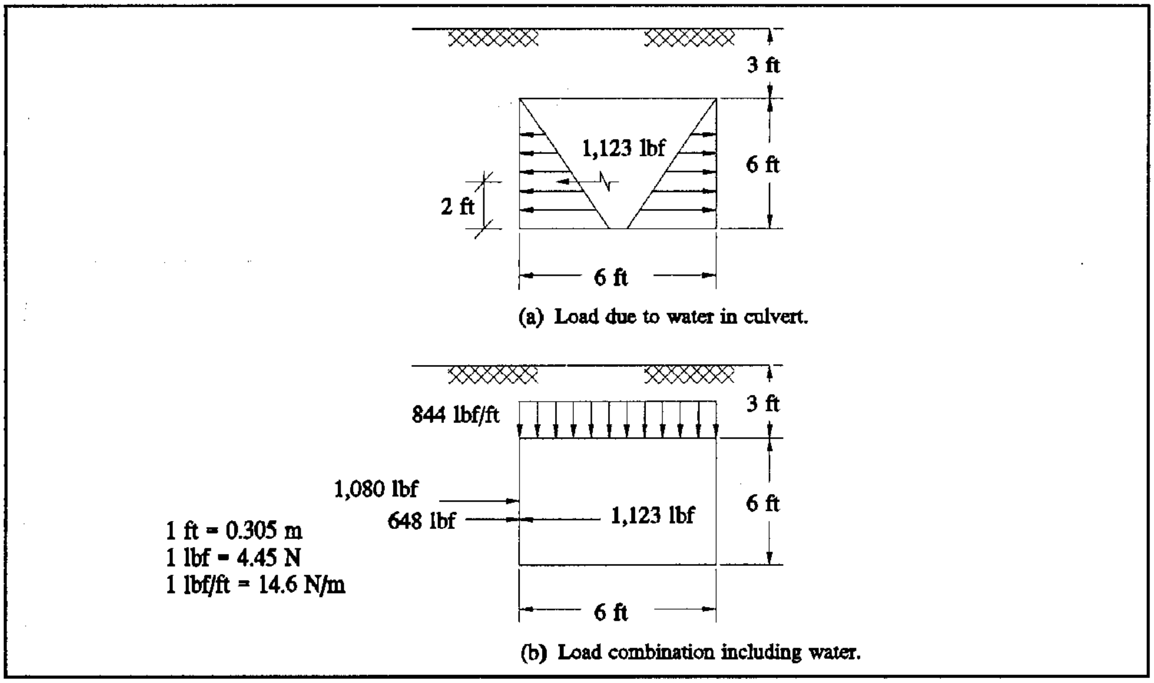

Figure 37. Load combinations in culvert.

The reaction force R_2 for the vertical member (figure 39) is:

$$R_2 = \frac{648\,lbf \times 2ft + 1{,}080\,lbf \times 3ft}{6ft} = 756\ lbf(3.36kN) \tag{65}$$

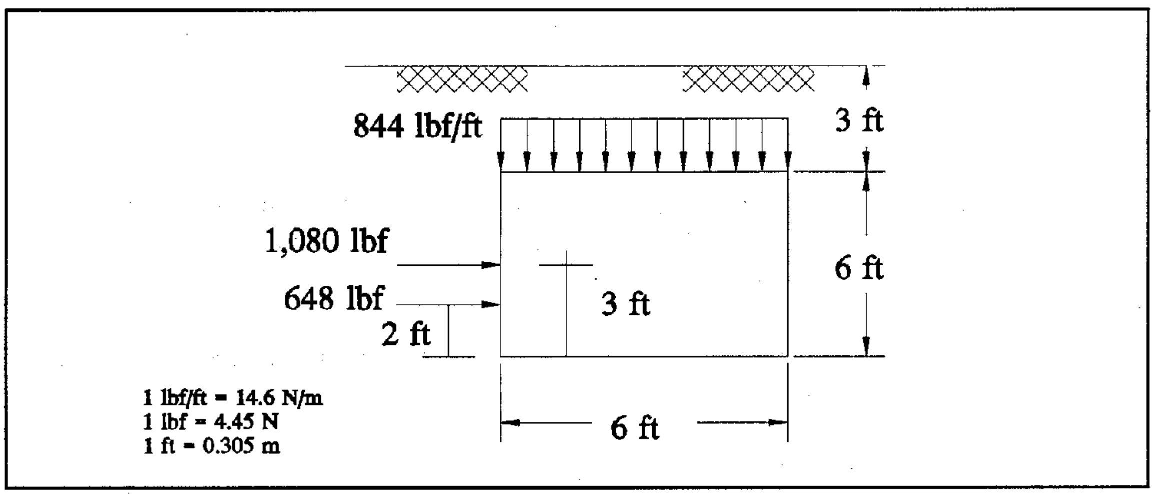

Figure 38. Total loads acting on culvert.

The total forces acting on the top 1-ft (0.305-m) section are shown in figure 40, and since bending occurs about the x-axis only, the interaction formula given by NDS 3.9.2 is written as:[16]

$$\left[\frac{f_c}{F'_c}\right]^2 + \frac{f_{b1}}{F'_{b1}\left[1-\left(\frac{f_c}{F_{cE1}}\right)\right]} \leq 1.0 \tag{66}$$

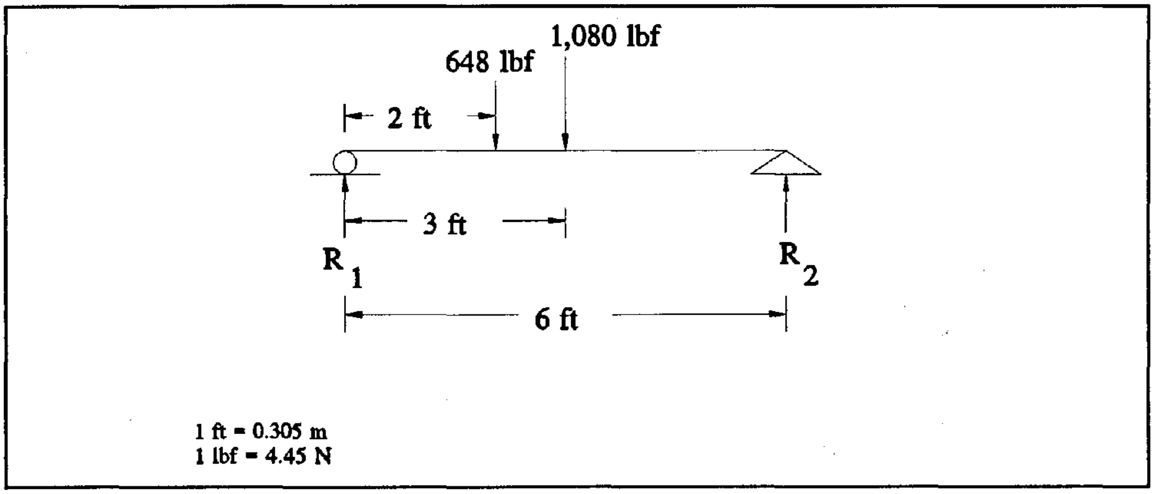

Figure 39. Reaction force for vertical member.

The axial stress is

f_c = 756 lbf/(12 in x 6.5 in) = 9.7 lbf/in^2 (67 kN/m^2)

The computation of F'_C should follow NDS 3.7.1., and is computed as

$F'_C = F^* \times C_p$

where,

F_C^*	=	Tabulated compression design value multiplied by all applicable adjustment factors except the C_p value
C_p	=	Column stability factor

On the basis of figure 40, the beam-column is assumed hinged, and therefore the effective length factor K_e = 1.0.

The effective length, l_{e1} = $K_e(l)$
= 1.0 x (6 ft x 12 in/ft) = 72 in (1,828 mm)

The slenderness ratio, l_{e1}/d = 72 in/6.5 in = 11

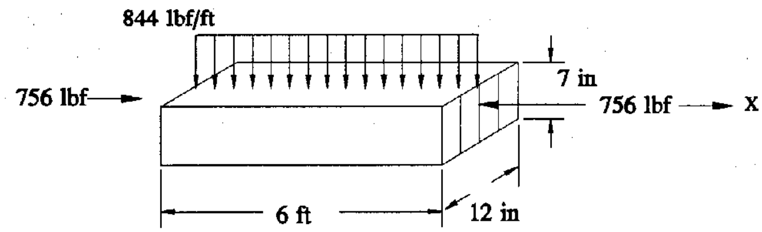

Figure 40. Total load for top member.

$$F_{CEI} = \frac{K_{CE}E'}{(l_{e1}/d_1)^2} = \frac{0.3 \times 1.62 \times 10^6 \; lbf/in^2}{(11)^2} = 4{,}016.5 \; lbf/in^2 \; (28MPa) \qquad \textbf{(67)}$$

where,

K_{CE} = 0.3 (NDS 3.7.1.5)

The column stability factor is given by

$$C_p = \frac{1+\alpha}{2c} - \sqrt{\left[\frac{1+\alpha}{2c}\right]^2 - \frac{\alpha}{c}} \qquad \textbf{(68)}$$

where,

$$\alpha = \frac{F_{CEI}}{F_C^*} \qquad \textbf{(69)}$$

c = 0.8 (for sawn timber)
F_C^* = 1,188 lbf/in^2 (8.18 MPa) (step #2)
α = 4016.5 lbf/in^2/1,188 lbf/in^2 = 3.38

$$C_p = \frac{1+3.38}{2(0.8)} - \sqrt{\left[\frac{1+3.38}{2(0.8)}\right]^2 - \frac{3.38}{0.8}} = 0.93 \qquad \textbf{(70)}$$

F'_C = F_C^* x C_p = 1,188 lbf/in^2 x 0.93 = 1,105 lbf/in^2 (7.6 MPa)

Computation of F'_{b1} should follow NDS 3.3. Since the culvert members are mechanically laminated, the beam stability factor, C_L, is equal to 1.0 (NDS 3.3.3). In addition, since the nominal depth-to-breadth ratio, d/b, is 2 to 1, no lateral bracing is required (see NDS 4.4.1). Therefore, the allowable bending strength is

F'_{b1} = 1,315 lbf/in^2 (9 MPa) (from step # 2)

The boundary conditions assumed are pin-pin, and the maximum moment $(M_x)_{max}$ is computed as $M_x = W_{TOTAL}\ L^2/8$.

$f_{b1} = M_x / S_x$
S_x = 12 in x (6.5 in)^2 / 6 = 84.5 in^3 (0.0014 m^3)
M_x = 844 lbf/ft x (6 ft)^2/8 = 3,798 lbf-ft = 45,576 lbf-in (5 kN-m)
f_{b1} = 45,576 lbf-in / 84.5 in^3 = 539 lbf/in^2 (3.5 MPa)

Substituting in equation 66,

$$\left[\frac{9.7\ lbf/in^2}{1{,}105\ lbf/in^2}\right]^2 + \frac{539\ lbf/in^2}{1{,}315\ lbf/in^2\left[1 - \dfrac{9.7\ lbf/in^2}{4{,}016.5\ lbf/in^2}\right]} \leq 1.0 \qquad \textbf{(71)}$$

$$0.0008 + 0.41 < 1.0 \qquad \text{OK}$$

Checking for beam-column action in the vertical member requires the determination of the axial load, i.e., the reaction forces on the top member (see figure 40).

The reaction force R_1 for the top member (figure 41) is:

$$R_1 = R_2 = \frac{W_{TOTAL}\,L}{2} = \frac{844\ lbf/ft \times 6ft}{2} = 2{,}532\ lbf\ (11.26\,kN) \qquad \textbf{(72)}$$

The total forces acting on the vertical member are shown in figure 42, and since bending occurs about the x-axis only, the interaction formula given by NDS 3.9.2 is reduced to equation 66.

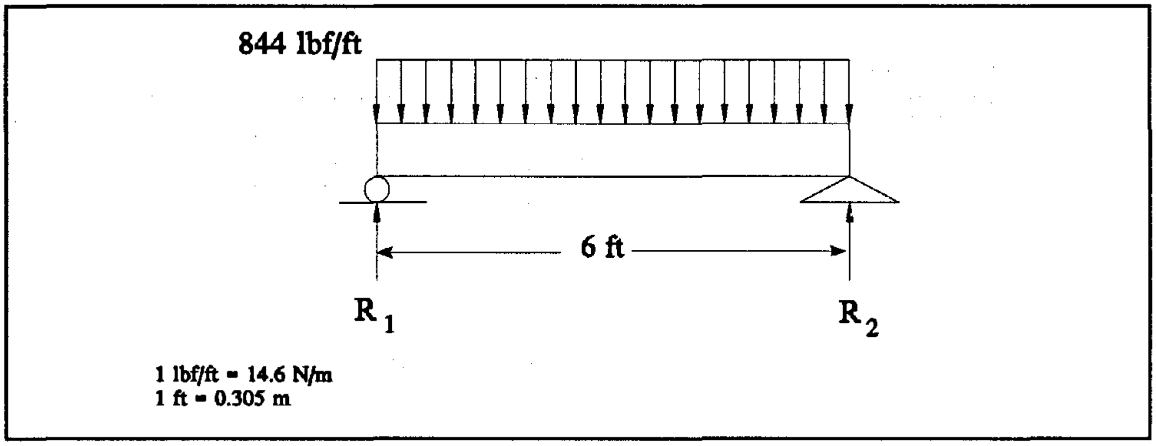

Figure 41. Reaction force for top member.

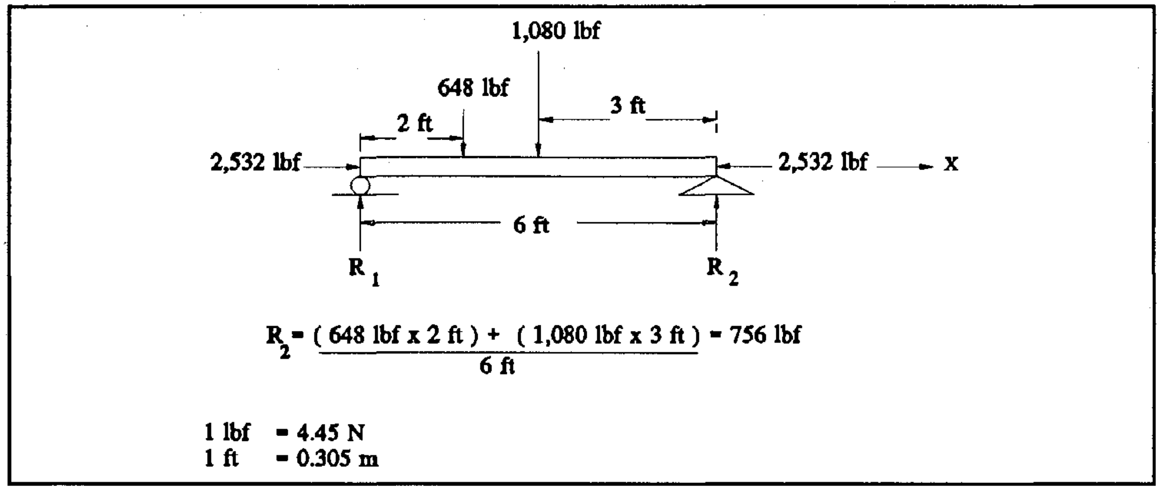

Figure 42. Total loads for vertical member.

The stresses are

f_c = 2,532 lbf/(12 in x 6.5 in) = 32 lbf/in^2 (0.22 MPa)
F'_C = 1,105 lbf/in^2 (7.28 MPa)
F'_{b1} = 1,315 lbf/in^2 (9 MPa) (from step #2)

Based on figure 42, the maximum moment $(M_x)_{max}$ is $M_x = R_2 \times 3$ ft.

M_x = 756 lbf x 3 ft = 2,268 lbf-ft = 27,216 lbf-in (3 kN-m)
$f_{b1} = M_x / S_x$
S_x = 84.5 in^3 (0.0014 m^3)
f_{b1} = 27,216 lbf-in / 84.5 in^3 = 322 lbf/in^2 (2.2 MPa)

Substituting in equation 66,

$$\left[\frac{32\ lbf/in^2}{1{,}105\ lbf/in^2}\right]^2 + \frac{322\ lbf/in^2}{1{,}315\ lbf/in^2\left[1-\frac{32\ lbf/in^2}{4{,}016.5\ lbf/in^2}\right]} \leq 1.0 \qquad \textbf{(73)}$$

$$0.02 + 0.24 = 0.26 < 1.0 \quad \text{OK}$$

5. Check shear at end notches.

From figure 43, the end reactions for the top member are computed as:

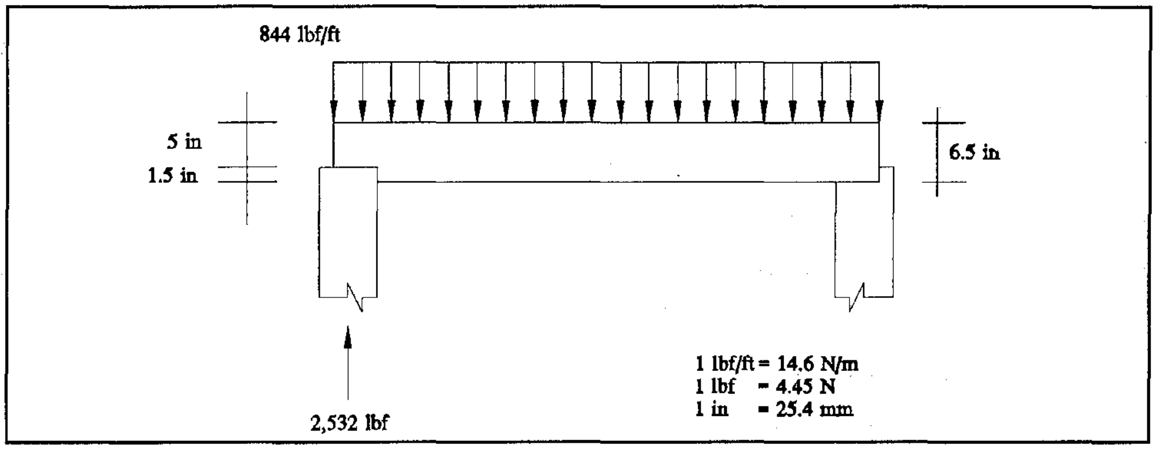

Figure 43. Shear at end notches.

$R_1 = R_2 = W_{TOTAL}\ L/2 = 2{,}532$ lbf (11.26 kN)

The horizontal shear stress is computed by (NDS 3.4.4.1):

$$f_v = \frac{3}{2}\ \frac{2{,}532\ lbf}{12\ in \times 5\ in}\left[\frac{6.5\ in}{5\ in}\right] = 82\ lbf/in^2\ (0.56\ MPa) \qquad \textbf{(74)}$$

$F'_v = 83$ lbf/in^2 (0.57 MPa) (see step # 2)
$f_v < F'_v$ OK

Since the top and side members are notched, the axial load acts with an eccentricity equal to 2.57 in (65 mm) (figure 44), developing a positive moment, which in turn, reduces the total moment acting on the top member. Therefore, the axial load was assumed to act concentrically.

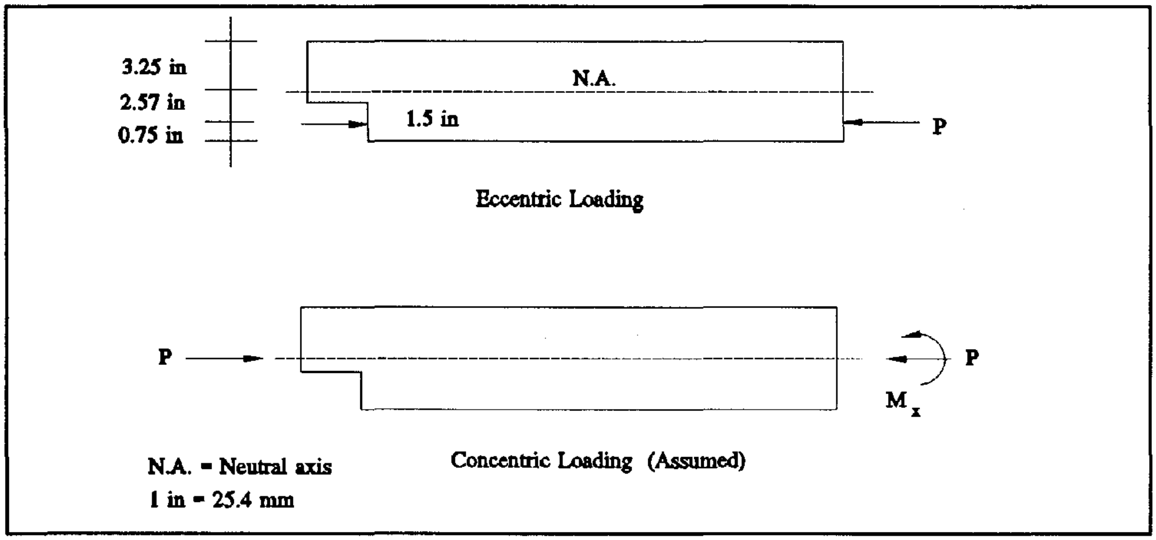

Figure 44. Eccentric loading.

By inspection, the most critical bending load is applied to the top member which is subjected to a uniformly distributed load, and therefore this member is checked for shear.

6. Check bearing.

Bearing parallel to grain: The actual bearing on the end grain of the member is computed as (figure 45):

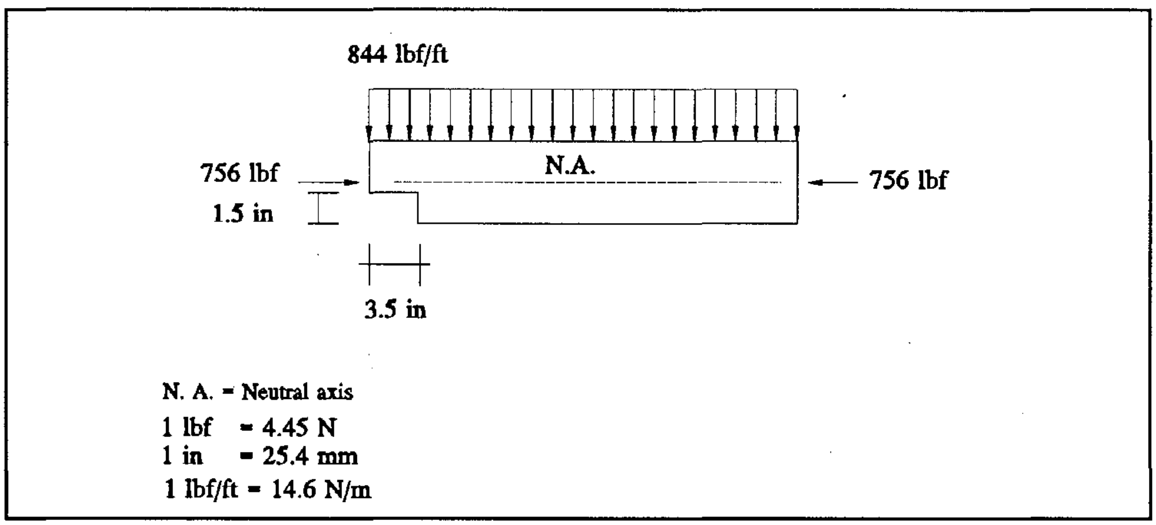

Figure 45. Bearing stress.

$$f_g = \frac{P}{A} \tag{75}$$

where,

f_g = Actual bearing stress parallel to grain, lbf/in^2 (kPa)
P = Total applied load, lbf (N)
A = Net bearing area, in^2 (mm^2)

$$f_g = \frac{P}{(d - d')b} = \frac{756\, lbf}{1.5\, in \times 12\, in} = 42\ lbf/in^2\, (0.3\, MPa) \tag{76}$$

F'_g = 1,818 lbf/in^2 (12.5 MPa) (step #2)
F'_g 0.75 = 1,818 lbf/in^2 x 0.75 = 1,363.5 lbf/in^2 (9.4 MPa)
$f_g < 0.75\ F'_g$ OK

By inspection, no metal bearing plates are needed (see NDS 3.10.1.3).[16]

Bearing perpendicular to grain: The actual bearing stress perpendicular to grain is given by:

$$f_{cp} = \frac{R}{A} \tag{77}$$

where,

f_{cp} = Actual stress in compression perpendicular to grain, lbf/in^2 (kPa)
R = Reaction or bearing force at the support, lbf (N)
A = Net bearing area, in^2 (mm^2)

$$f_{cp} = \frac{2{,}532\, lbf}{12\, in \times 3.5\, in} = 60.28\ lbf/in^2\, (0.41\, MPa) \tag{78}$$

F'_{cp} = 418.75 lbf/in^2 (2.88 MPa) (step #2)

$f_{cp} < F'_{cp}$ OK

7. Check deflection.

As a check, the deflection for a simply supported beam with a uniform load can be computed as:

$$\Delta = \frac{5 \times 70.4\ lb/in \times (6ft \times 12\ in/ft)^4}{384 \times 1.62 \times 10^6 \times 274.6\ in^4} = 0.055\ in\,(1.4\,mm) \qquad \textbf{(79)}$$

where,

E'	$= 1.62 \times 10^6$ lbf/in^2 (11,162 MPa) (step #2)
W_{TOTAL}	= 844 lbf/ft (12.3 kN/m) = 70.4 lbf/in (1.14 kN-m)
I	= 12 in x (6.5 in)^3/12 = 274.6 in^4 (11 x 10^{-5} m^4)
Δ	= 0.055 in = L/1,309 < L/800 OK

Such a small deflection justifies our approach in treating the design of this structure as a rigid culvert.

CHAPTER 5. TIMBER CRIB DESIGN

CONSIDERATIONS IN THE DESIGN OF TIMBER CRIB ABUTMENTS

A crib wall is essentially a gravity-type retaining wall made from timber, precast concrete, or metal. It consists of the following members (see figures 46 and 47):

1. Members forming the front face of the wall (called face stretchers or stringers).
2. Members forming the back of the wall (called back stretchers or anchors).
3. Members tying the front and back faces together (called headers, ties, or spacers).

The crib wall cells are connected to each other to extend the width of the required bridge abutment. The interior space of each cell is filled with granular material. The width, b, of the crib is determined as for an ordinary gravity wall (figure 47). Units should be chosen so that the width of the stretchers, f, is at least equal to or greater than twice the width of the header (2e) (see figure 48). Otherwise, dry granular fill will not be retained within the cell.[29] In figure 48, d is equal to zero.

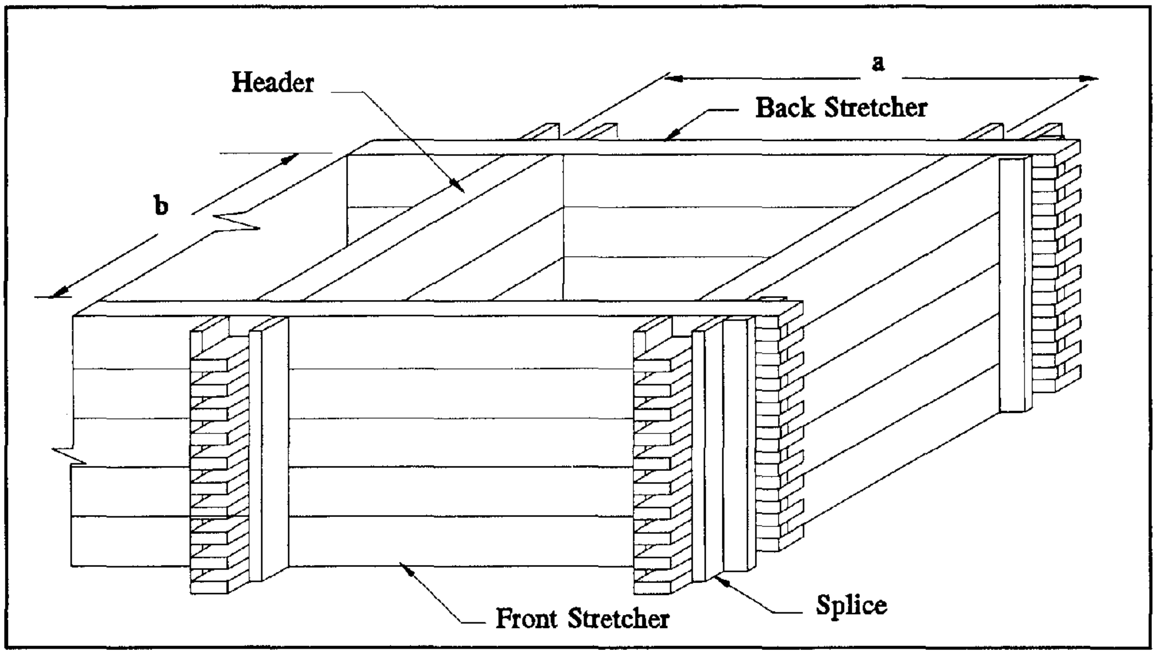

Figure 46. Single-cell crib abutment.

The horizontal earth pressure due to the soil filling inside the crib introduces bending stresses in the face stretchers. The reaction from the face stretchers are transmitted by headers to the back stretchers. The face stretchers spanning the distance, a, are designed to withstand the lateral bending (figure 47). The headers are designed to transmit the tensile reaction from the face stretchers and to support the weight of the superimposed loads.[19]

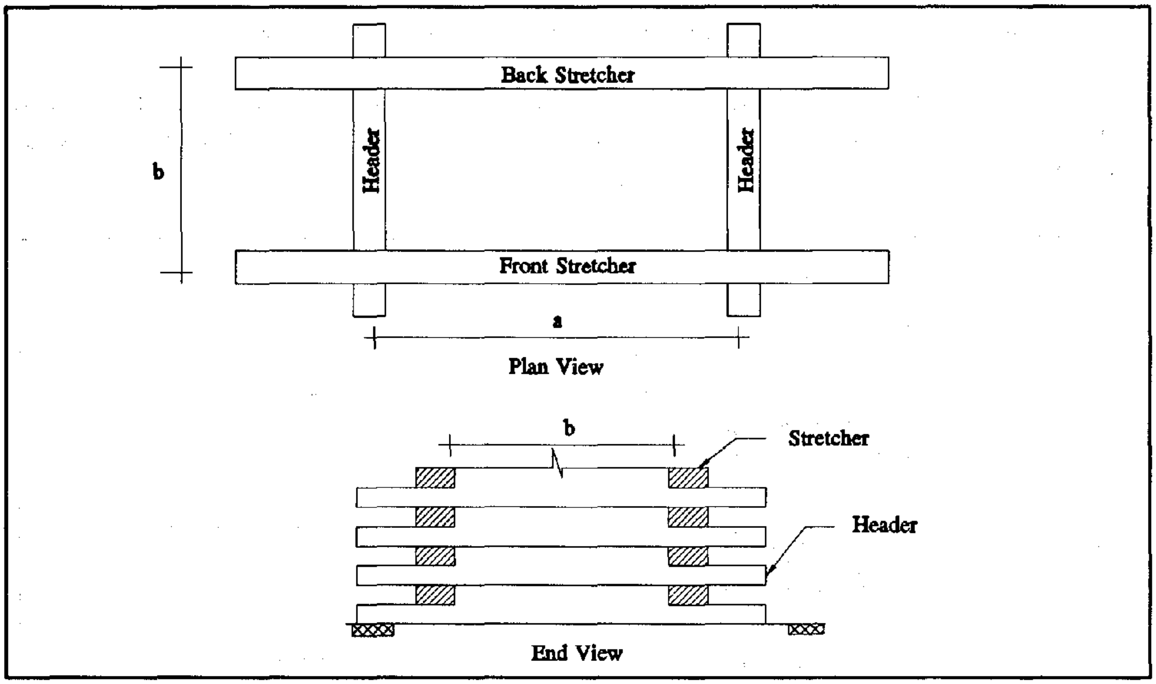

Figure 47. Crib wall.

The width of the single-cell crib wall is limited by the length of the header, which is customarily supplied in 6-ft (1.83-m) and 8-ft (2.44-m) lengths. An 8-ft (2.44-m) header limits the safe height of a single-cell crib wall to a maximum of about 16 ft (4.88 m).[29] For greater wall heights, interlocking twin-cell crib walls must be used. Twin walls in excess of 24 ft (7.32 m) in height, however, can create problems (such as outward bulge of the wall), since they can be very sensitive to transverse differential settlements.[29]

Loads on Timber Crib Abutments

Lateral Pressure Inside the Crib: The maximum lateral pressure occurs where height is equal to the width of the crib.[29]

$$P_1 = (d+e)(a)(K_o)(\gamma_s)(b) \qquad \textbf{(80)}$$

where,

d	= Void space between members (zero in figure 48)
e	= Width of header
a	= Span of stretcher
b	= Span of header
γ_s	= Unit weight of soil
K_o	= Coefficient of lateral earth pressure at rest = 0.5

This lateral pressure corresponds to that in a silo and will develop if the crib is filled with soil prior to backfilling. The active pressures after backfilling should not create bending moments exceeding those due to internal pressure.[29]

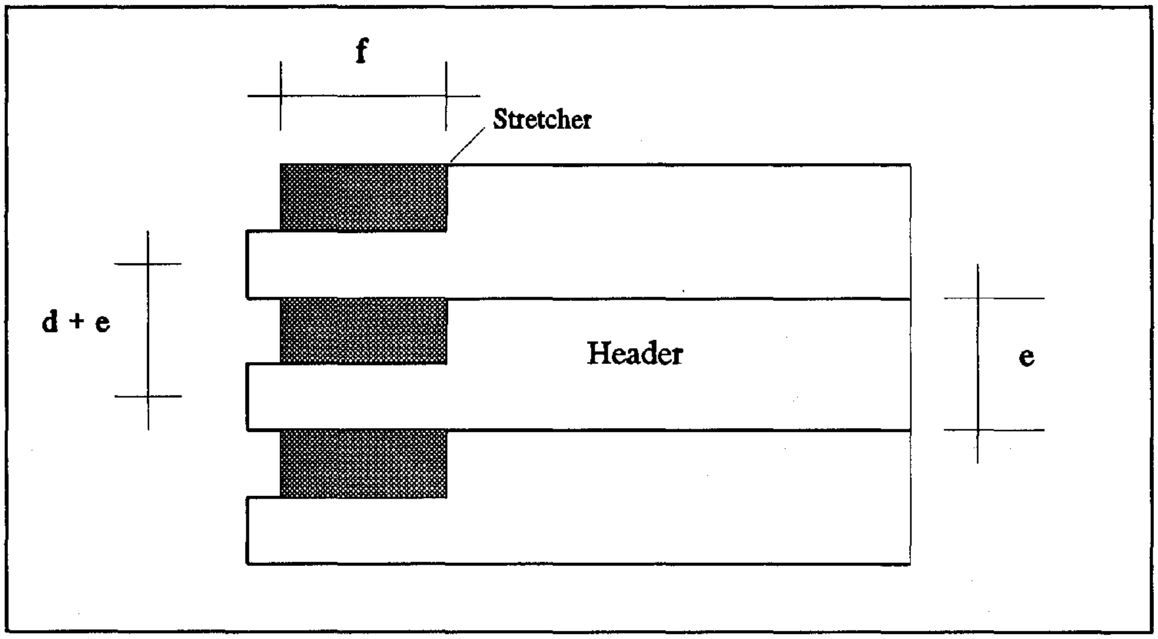

Figure 48. Crib wall detail.

Lateral Earth Pressures: Active lateral earth pressures act on the timber crib back walls and are a direct function of the vertical pressure. The equation for computing lateral earth pressure according to the Rankine theory is:[10]

$$P_a = \frac{1}{2}\gamma_s H^2 K_a \quad \textbf{(81)}$$

where,

P_a	= Active earth pressure
γ_s	= Unit weight of backfill soil
H	= Height of embankment
K_a	= Coefficient of active earth pressure ≈ 0.3

Dead Loads: Dead loads include the granular material inside the crib and the weight of the crib members. In addition, the weight of the bridge superstructure and timber cap should be included.

Live Loads: Live loads are based on AASHTO loading requirements for HS-20 or HS-25 truck loadings.[12]

DESIGN PROCEDURES

In this section we present a design procedure for a timber crib abutment. The details of this timber crib abutment are given in figures 46, 47, and 48. The system consists of stacked rectangular or square elements (stretchers and headers) to form cells that are filled with soil. The members are typically rough-sawn timbers of 10 in by 10 in (254 mm by 254 mm). To allow for end-bearing between the stretchers and headers, every member is notched at both ends (see figure 49).

The stretchers and headers are assumed to be simply supported (conservative), with a span extending from center-of-notch to center-of-notch. The proposed design guidelines for this type of crib abutment are presented next, followed by a design example.

1. **Define bridge geometry, loading, site conditions, and height of abutment.**

2. **Select a species and grade of lumber for crib abutment and determine the allowable design values given in NDS.**

   Timber cribs are usually constructed from lumber in the *Posts and Timbers* size classification. Select a species and grade of lumber from NDS table 4D. Generally, grade no. 1 material or better is selected, and the following allowable design values are computed:[16]

Bending strength:	$F_b' = F_b C_m C_F$
Elastic modulus:	$E_L' = E_L C_m$
Compression perpendicular to grain:	$F_{cp}' = F_{cp} C_m$
Shear strength:	$F_v' = F_v C_m$

   where,

   C_m = Moisture content factor, which has a distinct value *for each design property* given in NDS

   C_F = Size factor from NDS table 4D

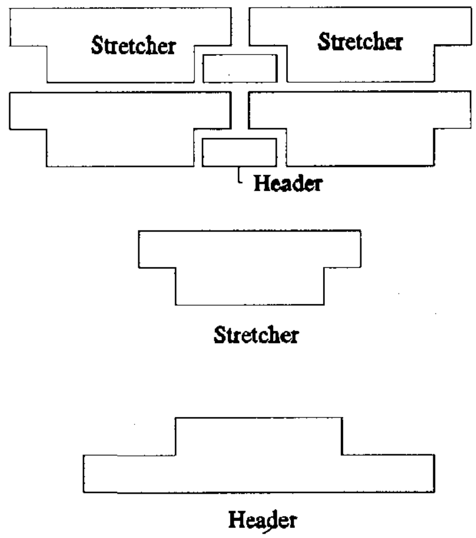

Figure 49. Stretchers and headers.

3. Timber cap design.

Design the timber cap following the procedures outlined in example I in the timber piles chapter.

4. Header design.

Lateral Bending (about weak axis): The horizontal earth pressure due to the soil-fill inside the crib induces bending stresses in the headers. The maximum lateral force is computed from equation 80.

The lateral bending stress can be computed from

$$f_b = \frac{M_y}{S_y} \tag{82}$$

where,

M_y = Moment about y-axis (weak axis) due to lateral earth pressure
S_y = Section modulus about the y-axis

Bending about Strong Axis: Assume the header is simply supported and compute the maximum bending moment due to vertical live and dead loads. Compute the induced bending stress about the x-axis in a similar manner as for lateral bending.

Bearing Perpendicular to Grain: The actual bearing stress perpendicular to grain is computed as

$$f_{cp} = \frac{R}{A} \tag{83}$$

where,

f_{cp} = Actual stress in compression perpendicular to grain
R = Reaction or bearing force at support
A = Net bearing area

5. Stretcher design.

Lateral Bending (about weak axis): The horizontal earth pressure due to the soil-fill inside the crib induces bending stresses in the face stretchers. The maximum lateral force is computed from equation 80.

The lateral bending stress can be computed from

$$f_b = \frac{M_y}{S_y} \tag{84}$$

where,

M_y = Maximum lateral bending moment = $WL^2/8$
S_y = Section modulus

Bearing Perpendicular to Grain: The actual bearing stress perpendicular to grain is computed as

$$f_{cp} = \frac{R}{A} \tag{85}$$

where,

f_{cp} = Actual stress in compression perpendicular to grain
R = Reaction or bearing force at support
A = Net bearing area

6. Splice design.

A bolt connection is used to hold the stretcher and header in place to prevent outward movement (figure 50).

Bolt Load: The allowable bolt design value is computed as

$$Q' = Q\ C_D\ C_M \tag{86}$$

where,
Q' = Allowable bolt design value
Q = Tabulated design value (AITC table 6-20)[5]
C_D = Load duration factor
C_M = Moisture content factor

Member Capacity: The allowable stress in tension-parallel to grain is computed as

$$f_t = \frac{Applied\ Load}{A_n} \tag{87}$$

where,
A_n = Net area minus hole diameter

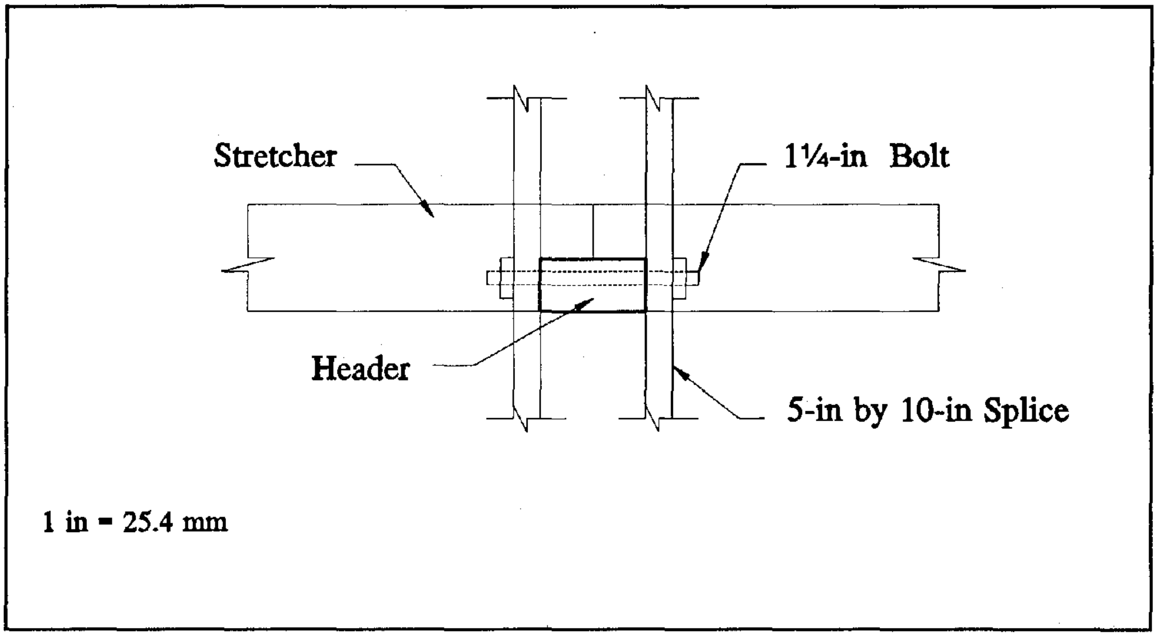

Figure 50. Connection detail between stretcher and header.

7. Stability analysis.

Common procedures in abutment design involve stability checks against movement. Essentially, there are three means by which an abutment can move: (1) horizontally (by sliding), (2) vertically (bearing capacity failure), and (3) rotating (by overturning). The standard procedure is to check for stability with respect to each of the three movements to ensure that an adequate factor of safety is present.

The factor of safety against sliding is found by dividing the sliding resistance force by the sliding force. The sliding resistance force is the product of the total normal force and the coefficient of friction (μ) between the base of the crib abutment (usually reinforced concrete footing) and the underlying soil. The sliding force is typically the horizontal component of the lateral earth pressure exerted against the abutment by the backfill soil.

The factor of safety against overturning is determined by dividing the total righting moment by the total overturning moment. Righting moments and overturning moments are computed about the abutment's toe.

The factor of safety against bearing capacity failure is determined by dividing the ultimate bearing capacity by the actual contact base pressure.[30]

The three factors of safety with regard to stability analysis are:

$$(F.S.)_{sliding} = \frac{Sliding\ Resistance\ Force}{Sliding\ Force} \tag{88}$$

$$(F.S.)_{overturning} = \frac{Total\ Righting\ Moment}{Total\ Overturning\ Moment} \tag{89}$$

$$(F.S.)_{bearing\ capacity\ failure} = \frac{Soil\ Ultimate\ Bearing\ Capacity}{Base\ Pressure} \tag{90}$$

Some common factors of safety for sufficient stability are:

$(F.S.)_{sliding}$ = 1.5
$(F.S.)_{overturning}$ = 2.0 (cohesive backfill soil)
$(F.S.)_{bearing\ capacity\ failure}$ = 3.0

For further information on stability analysis, see references 20, 29, and 30.

DESIGN EXAMPLE

1. Define bridge geometry, loading, and site conditions.

Stress-Laminated Timber Deck:

Span of superstructure:	20 ft (6.1 m)
Width of superstructure:	24 ft (7.32 m)
Width of abutment:	26 ft (7.93 m)
Loading:	HS-20

Assumptions and Design Data:

Height of abutment:	20 ft (6.1 m)
d = Void spacing:	0
e = Width of header:	10 in (254 mm)
a = Stretcher length:	6.5 ft (1.98 m)
b = Header length:	4.625 ft (1.41 m)
Timber cap:	16 in by 16 in by 26 ft (406 mm by 406 mm by 7.9 m)

γ_s	= Unit weight of soil:	100 lbf/ft^3 (15.7 kN/m^3)
K_o	= Coefficient of lateral earth pressure:	0.5
μ	= Coefficient of friction:	0.58[29]
The foundation soil's ultimate bearing capacity:		6.5 ton/ft^2 (623 kPa)

2. Select species and grade of lumber, and determine allowable values.

Material: Douglas fir-larch of grade dense no. 1 (NDS table 4D)

$$F_b' = F_b\, C_m$$
$$= 1{,}400 \text{ lbf/in}^2 \times 1.0 = 1{,}400 \text{ lbf/in}^2 \quad (9.64 \text{ MPa})$$

$$E_L' = E_L\, C_m$$
$$= 1.7 \times 10^6 \text{ lbf/in}^2 \times 1.0 = 1.7 \times 10^6 \text{ lbf/in}^2 \quad (11{,}713 \text{ MPa})$$

$$F_{cp}' = F_{cp}\, C_m$$
$$= 730 \text{ lbf/in}^2 \times 0.67 = 489 \text{ lbf/in}^2 \quad (3.36 \text{ MPa})$$

$$F_v' = F_v\, C_m$$
$$= 85 \text{ lbf/in}^2 \times 1.0 = 85 \text{ lbf/in}^2 \quad (0.58 \text{ MPa})$$

$$F_t' = F_t\, C_m$$
$$= 950 \text{ lbf/in}^2 \times 1.0 = 950 \text{ lbf/in}^2 \quad (6.54 \text{ MPa})$$

3. Timber cap design.

Timber Cap Dimensions: 16 in by 16 in by 26 ft (406.4 mm by 406.4 mm by 7.9 m)

Material Properties:

Douglas fir-larch grade #1. The allowable design values given in NDS table 4D (NDS 1991) are:

Bending strength: $F_b' = F_b \times C_m \times C_F$
$= 1{,}200\ \mathrm{lbf/in^2} \times 1.0 \times 0.98$
$= 1{,}176\ \mathrm{lbf/in^2}$ (8 MPa)

Compression perpendicular to grain: $F_{cp}' = F_{cp}' \times C_m$
$= 625\ \mathrm{lbf/in^2} \times 0.67$
$= 418\ \mathrm{lbf/in^2}$ (2.79 MPa)

Shear strength: $F_v' = F_v \times C_m$
$= 85\ \mathrm{lbf/in^2} \times 1.0$
$= 85\ \mathrm{lbf/in^2}$ (0.57 MPa)

where,
C_m = Moisture content factor
C_F = Size factor

Dead and live load

Maximum dead and live loads on each abutment.

Dead load of superstructure = 48,184 lbf (214 kN)
Weight of cap = (16/12 ft) x (16/12 ft) x 26 ft x 58 lbf/ft^3 = 2,680 lbf (12 kN)

Total dead load on each abutment: (48,184 lbf/2) + 2,680 lbf = 26,772 lbf (119 kN)
Total dead load per unit width: 26,772 lbf/26 ft = 1,029 lbf/ft (15 kN/m)
Live load on abutment:

The distribution width, D, is:[3]

$$D = [\, b_f + (2 \times d)\,] \times C_b \qquad \mathbf{(91)}$$

where,
D = Transverse wheel load distribution width, in (mm)
P_w = Wheel load, HS-20
= 16,000 lbf (71.2 kN)

b_f = Truck tire width perpendicular to traffic = $\sqrt{0.025\ P_w}$ in (mm).
= $(0.025P_w)^{0.5}$ = $(0.025 \times 16{,}000\ \text{lbf})^{0.5}$ = 20 in (508 mm)
d = Deck thickness = 14 in (356 mm)
C_b = Butt joint factor

Butt joint pattern: 1 in 5

$$C_b = \frac{j}{j+1} \tag{92}$$

where,
j = Number of laminae between two successive butt joints
= 4
C_b = 4/5 = 0.8
D = (20 in + 28 in) x 0.8 = 38.4 in (975 mm)

The live load is distributed over a portion of the deck equal to 38.4 in = 3.2 ft (0.97 m). The live load for a single lane is 20,800 lbf (92.5 kN). Therefore, the live load distribution is 20,800 lbf/3.2 ft = 6,500 lbf/ft (95 kN/m)

The crib abutment consists of four cells of 6.5 ft by 4.625 ft (2 m by 1.41 m) each (figure 51). Designing for a critical section of the cap (figure 51):

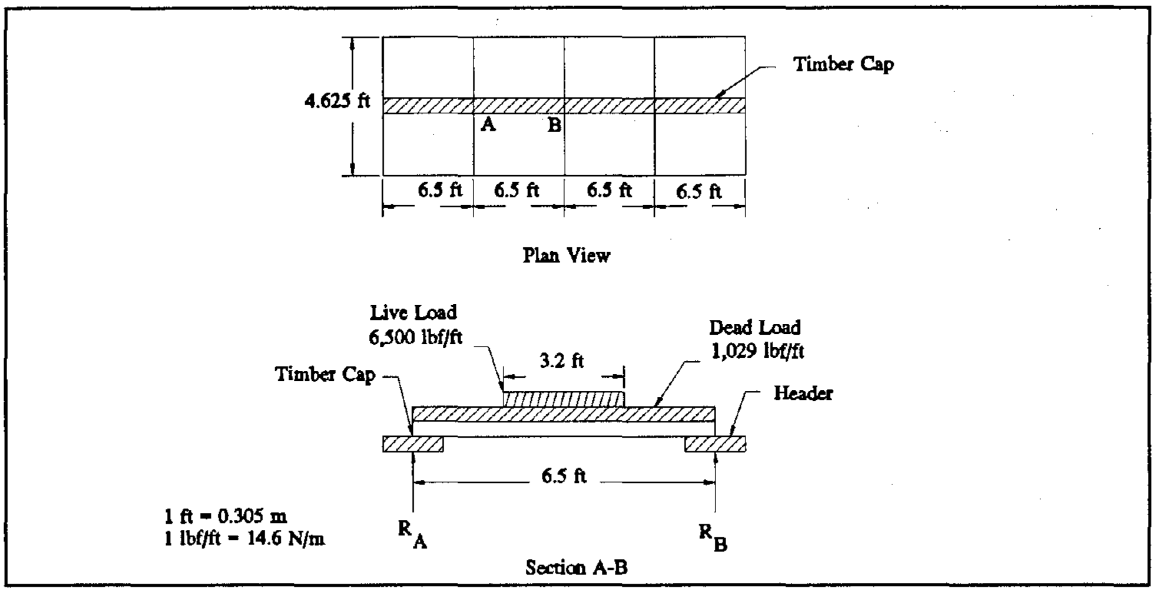

Figure 51. Cap section.

$R_A = R_B = [(1{,}029 \text{ lbf/ft} \times 6.5 \text{ ft}) + (6{,}500 \text{ lbf/ft} \times 3.2 \text{ ft})] / 2 = 13{,}744 \text{ lbf } (61 \text{ kN})$

Design checks for horizontal shear, bending stresses, and bearing follow the procedures given in example I for timber piles.

4. **Header design.**

Lateral bending: Maximum lateral force due to earth pressure is

$$P_1 = (e)(a)(K_o)(\gamma_s)(b) \tag{93}$$

$$P_1 = \frac{10\ in}{12} \times 6.5\,ft \times 0.5 \times 100\ lbf/ft^3 \times 4.625\,ft = 1{,}252\ lbf\ (5.57\ kN) \tag{94}$$

Lateral uniform load on header:

$$\frac{1{,}252\ lbf}{4.625\,ft} = 271\ lbf/ft \quad (4\ kN/m) \tag{95}$$

Maximum lateral moment (M_y):

$$M_y = \frac{271\ lbf/ft \times (4.625\,ft)^2}{8} = 725\ lbf\text{-}ft\ (8{,}700\ lbf\text{-}in)\ (983\ N\text{-}m) \tag{96}$$

Lateral bending stress:

$f_b = M_y/S_y$

$S_y = bd^2/6 = 9.5 \text{ in } (9.5 \text{ in})^2/6 = 143 \text{ in}^3 \ (2.3 \times 10^{-3} \text{ m}^3)$
$f_b = 8{,}700 \text{ lbf-in}/143 \text{ in}^3 = 60.8 \text{ lbf/in}^2 \ (0.42 \text{ MPa})$

$F_b' > f_b$ OK

Bearing on header from timber cap:

Assuming the timber cap is placed at the center of the abutment, bearing on the headers should be checked (see figure 52).

Designing for a critical section of the timber cap (see chapter 2 of this booklet for timber cap design):

$R_A = 13{,}744$ lbf (61 kN)

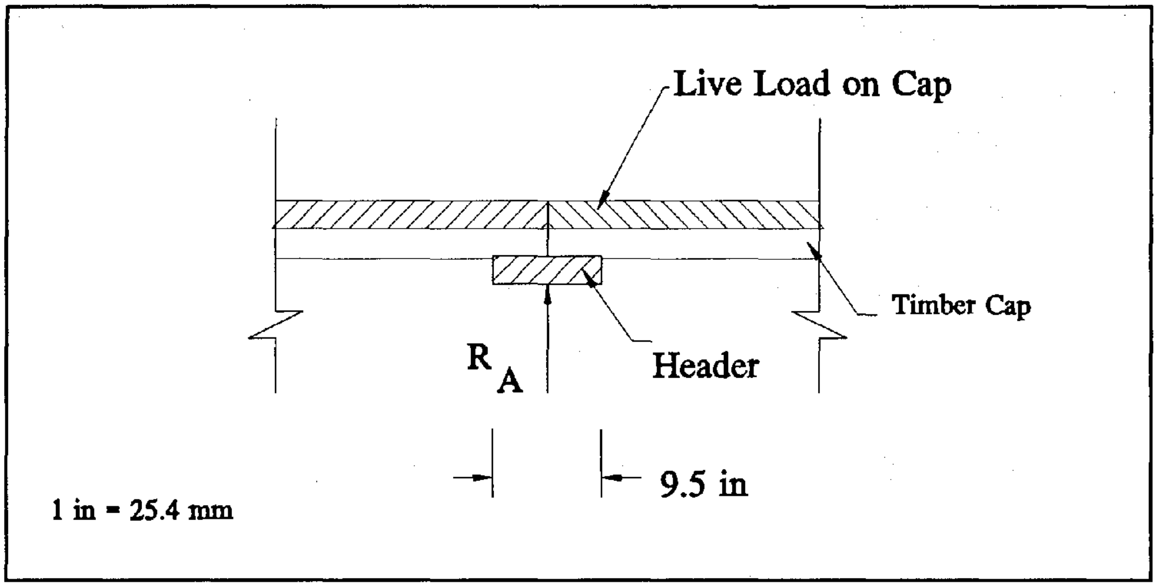

Figure 52. Bearing on headers.

Required bearing area = (13,744 lbf)/489 lbf/in^2 = 28.1 in^2 (18,381 mm^2)
Required bearing length = 28.1 in^2/9.5 in = 3 in (76.2 mm)

3 in (76.2 mm) < 4.5 in (114.3 mm) available OK

Bending about the strong axis (x-axis):

Assuming the header is simply supported (see figure 53):

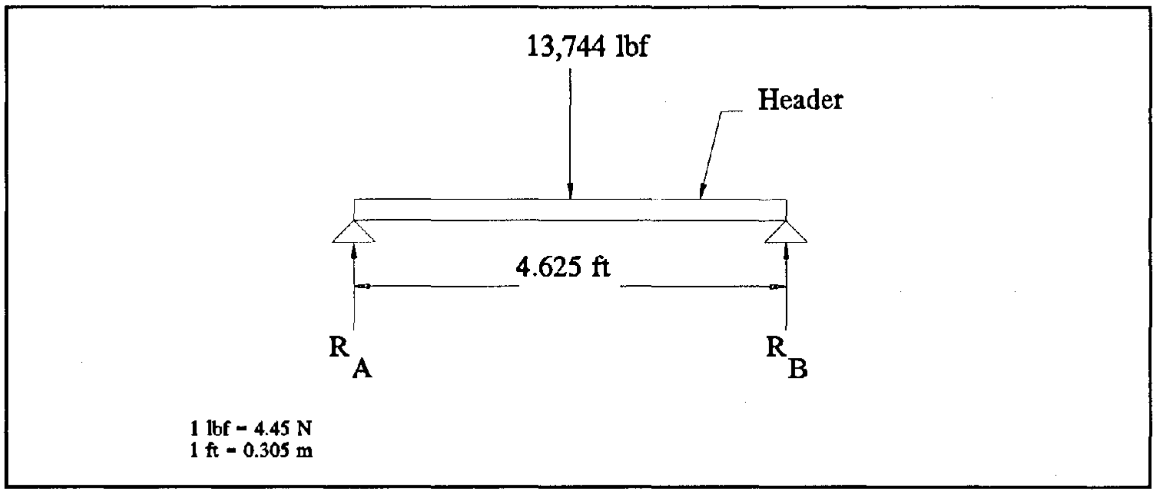

Figure 53. Bending about x-axis.

M_x = PL/4 = (13,744 lbf) x 4.625 ft/4 = 15,892 lbf-ft = 190,698 lbf-in (21 kN-m)

$f_{bx} = M_x/S_x$

$S_x = bd^2/6 = 9.5$ in $(9.5 \text{ in})^2/6 = 143 \text{ in}^3$ $(2.3 \times 10^{-3} \text{ m}^3)$
$f_{bx} = 190{,}698$ lbf-in/143 in^3 = 1,333 lbf/in^2 (9.18 MPa)

$F'_b = 1{,}400$ lbf/in^2 (9.64 MPa) $> f_{bx} = 1{,}333$ lbf/in^2 (9.18 MPa) OK

5. Stretcher design.

Lateral bending: Maximum lateral force due to earth pressure is

$$P_1 = (d+e)(a)(K_o)(\gamma_s)(b) \tag{97}$$

$$P_1 = \frac{10\ in}{12} \times 4.625\,ft \times 6.5\,ft \times 100\ lbf/ft^3 \times 0.5 = 1{,}252\ lbf\ (5.57\ kN) \tag{98}$$

Lateral uniform load on stretcher:

$$\frac{1{,}252\ lbf}{6.5\ ft} = 193\ lbf/ft \quad (2.8\ kN/m) \tag{99}$$

Maximum lateral moment (M_y):

$$M_y = \frac{193\ lbf/ft \times (6.5\,ft)^2}{8} = 1{,}019\ lbf\text{-}ft\ (12{,}231\ lbf\text{-}in)\ (1.3\ kN\text{-}m) \tag{100}$$

Lateral bending stress:

$f_b = M_y/S_y$
$S_y = bd^2/6 = 9.5$ in $(9.5 \text{ in})^2/6 = 143 \text{ in}^3$ $(2.3 \times 10^{-3} \text{ m}^3)$
$f_b = 12{,}231$ lbf-in/143 in^3 = 85.5 lbf/in^2 (0.58 MPa)

$F_b' > f_b$ OK

Bearing on stretchers from headers:

From figure 54:

$R_A = 13{,}744$ lbf (61 kN)
$f_{cp} = R_A/A$
$f_{cp} = 13{,}744$ lbf/(9.5 in x 4.5 in)= 321 lbf/in^2 (2.19 MPa)
$F'_{cp} = 489$ lbf/in^2 (3.36 MPa) $> f_{cp} = 321$ lbf/in^2 (2.19 MPa) OK

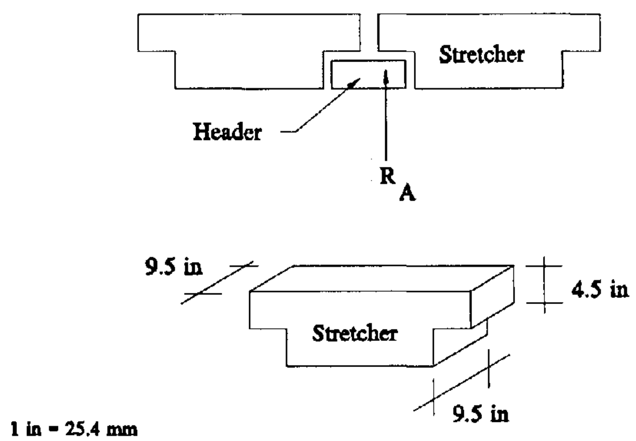

Figure 54. Bearing on stretcher.

6. Splice design.

Since the width of the header to support the stretcher is 10 in (254 mm), the bolt length is equal to the header width plus twice the splice thickness (figure 50). A single bolt is used, which is loaded perpendicular to the grain of the splice.

Load on bolt:

Lateral load on stretcher: 1,252 lbf (5.5 kN). Half of the load becomes a tension load applied to the header. An interior header is subjected to tension by the two adjacent stretchers.

Applied load on connection: (1,252 lbf/2) x 2 = 1,252 lbf (5.5 kN)
Use a 5-in by 10-in (127-mm by 254-mm) splice.

The connection is in double shear, and the load is applied perpendicular to the grain of the splice. The main header member is twice the thickness of the side splice members (figure 50).

For a 1¼-in (31.75-mm) diameter bolt having a length of 9.5 in (241.3 mm) in the main member (header), the allowable design value is

$$Q' = Q\ C_D\ C_M \qquad \textbf{(101)}$$

where,
Q' = Allowable bolt design value
Q = 4,510 lbf (20 kN) (AITC table 6-20)[5]
C_D = 0.9

C_M = 0.75 (NDS table 7.3.3)

$$Q' = 4{,}510\ lbf \times 0.9 \times 0.75 = 3{,}044\ lbf\ (13.5\ kN) \tag{102}$$

Q′ = 3,044 lbf (13.5 kN) > applied load = 1,252 lbf (5.5 kN) OK

Member Capacity: The capacity of the connection depends on the net area of the main member. Assuming that the bolt holes are 1/16 in (1.58 mm) larger than the bolt diameter, the net area of the member equals

A_n = (9.5 in x 4.5 in) - [(1.25 in + 1/16 in)^2 x 3.14/4] = 41.39 in^2 (26,695 mm^2)

The allowable stress in tension-parallel to grain is computed as

$$f_t = \frac{1{,}252\ lbf}{41.39\ in^2} = 30.24\ lbf/in^2\ (0.2\ MPa) \tag{103}$$

F'_t = 950 lbf/in^2 (6.54 MPa) > f_t = 30.24 lbf/in^2 (0.2 MPa) OK

7. Stability analysis.

A check of the stability of the crib abutment is performed by determining the factors of safety against: (1) sliding, (2) overturning, and (3) bearing capacity failure. The design calculations are for a 1-ft (0.305-m) length of wall (see figure 55).

Calculation of Righting Moments

Normal Force:

1. Weight of footing:
   17.25 ft x 18 in/12 x 150 lbf/ft^3 = 3,881 lbf/ft (56.6 kN/m)

2. Area of wall:
   2 (front and back stretchers) x 12 (units) x 9.5 in (height) x 9.5 in (thick) = 2,166 in^2 (1.4 x 10^6 m^2)

3. Weight of wall:
   (2,166 in^2/144) x 58 lbf/ft^3 = 872 lbf/ft (12.7 kN/m)

4. Weight of timber cap:
   (16 in/12) x (16 in/12) x 58 lbf/ft^3 = 103 lbf/ft (1.5 kN/m)

5. Height of backfill soil: 20 ft (6.1 m)

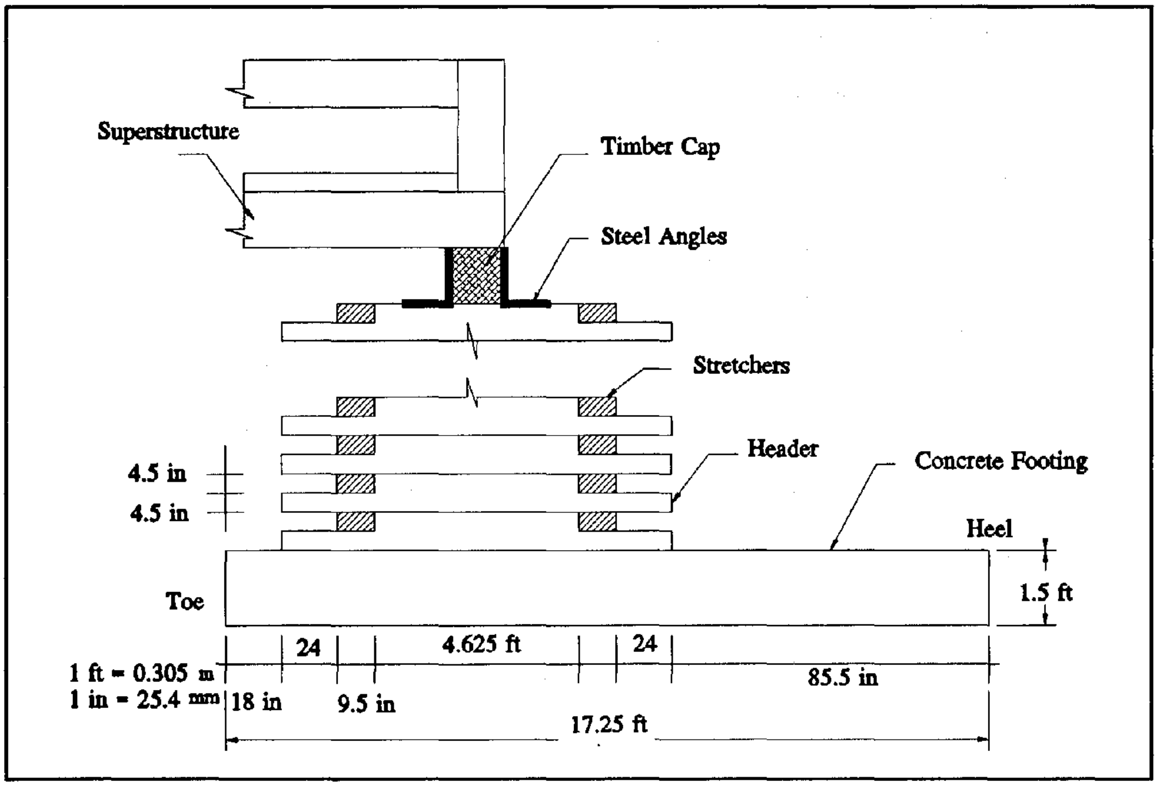

Figure 55. Stability analysis.

6. Area of soil at heel:
   (85.5 in + 24 in) x 20 ft = 183 ft^2 (17 m^2)

7. Weight of backfill soil:
   183 ft^2 x 100 lbf/ft^3 = 18,300 lbf/ft (267 kN/m)

8. Area of soil inside cell (see figure 55):
   [20 ft - 1.5 ft (concrete footing)] x 4.625 ft = 85.56 ft^2 (7.9 m^2)

9. Weight of soil inside crib:
   85.56 ft^2 x 100 lbf/ft^3 = 8,556 lbf/ft (125 kN/m)

10. Total normal force:

    103 lbf/ft + 18,300 lbf/ft + 872 lbf/ft + 8,556 lbf/ft = 27,831 lbf/ft (406 kN/m)

Righting Moment: Taking moments about toe (figure 55):

1. Moment due to footing:
   (3,881 lbf/ft) x (17.25 ft/2) = 33,473 lbf-ft/ft (45 kN-m/m)

2. Moment due to cap:
(103 lbf/ft) x (2 ft + 24 in/12 + 18 in/12) = 566.5 lbf-ft/ft (767 N-m/m)

3. Moment due to front wall:
(872 lbf/ft/2) x (18 in + 14.5 in + 9.5 in/2)/12 = 1,352 lbf-ft/ft (2 N-m/m)

4. Moment due to back wall:
(872 lbf/ft/2) x [18 in/12 + 24 in/12 + 4.75 in/12 + 4.625 ft]
= 3,714 lbf-ft/ft (5 N-m/m)

5. Moment due to soil inside crib:
(8,556 lbf/ft) x (42 in/12 + 2.31 ft) = 49,710 lbf-ft/ft (67 kN-m/m)

6. Moment due to soil at heel:
(18,300 lbf/ft) x (9.625 ft + 4.56 ft) = 259,494 lbf-ft/ft (351 kN-m/m)

7. Total righting moment:
33,473 lbf/ft + 566.5 lbf/ft + 1,352 lbf/ft + 3,714 lbf/ft + 49,710 lbf/ft + 259,494 lbf/ft = 348,310 lbf/ft (472 kN-m/m)

Calculation of Overturning Moment:

1. Moment due to backfill soil:

Resultant force due to active earth pressure is

$$P_a = \frac{1}{2} \gamma_s H^2 K_a \tag{104}$$

P_a = ½ x 100 lbf/ft^3 x (21.5 ft)^2 x 0.3 = 6,933.75 lbf/ft (101 kN/m)

Thus, the moment due to active earth pressure is
(6,933.75 lbf/ft) x (21.5 ft/3) = 49,645 lbf-ft/ft (67 kN-m/m)

2. Moment due to 2 ft (0.61) live load surcharge (AASHTO 3.20.3):

Resultant force due live load surcharge is:
P = 0.3 x 100 lbf/ft^3 x 2 ft x 21.5 ft = 1,290 lbf/ft (18.8 kN/m)

Moment due to surcharge:
(1,290 lbf/ft) x (21.5 ft/2) = 13,222 lbf-ft/ft (18 kN-m/m)

3. Total overturning moment:
49,645 lbf-ft/ft + 13,222 lbf-ft/ft = 62,867 lbf-ft/ft (85 kN-m/m)

Check for Sliding

$$(F.S.)_{sliding} = \frac{\mu \times Normal\ Force}{Horizontal\ Soil\ Force} \tag{105}$$

Normal force = 27,831 lbf/ft (406 kN/m)
$\mu = 0.58$
Horizontal soil force = 6,934 lbf/ft + 1,290 lbf/ft = 8,224 lbf/ft (120 kN/m)

$(F.S.)_{sliding}$ = (0.58) (27,831 lbf/ft)/(8,224 lbf/ft) = 1.96 > 1.5 OK

Check for Overturning

$(F.S.)_{overturning}$ = Total righting moment/Total overturning moment

$(F.S.)_{overturning}$ = (348,310 lbf/ft)/(62,867 lbf/ft) = 5.5 > 1.5 OK

Base Pressure Calculations

Location of resultant (R):

R = Total normal force (ΣV) if R acts x-bar from toe.

$$\bar{x} = \frac{\Sigma M_{toe}}{\Sigma V} = \frac{\Sigma M_R - \Sigma M_o}{\Sigma V} = \frac{348{,}310\ lb/ft - 62{,}867\ lb/ft}{27{,}831\ lb/ft} = 10.25\ ft\ (3.12\ m) \tag{106}$$

The resultant force must act within the middle third of the base.
Middle third = 17.25 ft/3 = 5.75 ft (1.75 m) < 10.25 ft (3.1 m) < 11.5 ft (3.5 m)

OK

Check for bearing capacity

Using the flexural formula:

$$\sigma_{\max} = \frac{Q}{A} \pm \frac{M_x y}{I_x} \pm \frac{M_y x}{I_y} \tag{107}$$

Q = Dead load and live load + weight of abutment
Q = 4,230 lbf/ft + 27,831 lbf/ft = 32,061 lbf/ft (468 kN/m)

A = 1 ft x 17.25 ft = 17.25 ft^2 (1.6 m^2)

M_x = 0 (one-way bending)

M_y = Q e

e = Eccentricity of resultant force, Q, found by taking moments about toe.

Location of resultant force (considering dead + live loads):

Total righting moment:
348,310 lbf-ft/ft + (4,230 lbf/ft x 5.5 ft) = 371,575 lbf-ft/ft (503 kN-m/m)

Total overturning moment:
62,867 lbf-ft/ft (85 kN-m/m)

Total normal force:
27,831 lbf/ft + 4,230 lbf/ft = 32,061 lbf/ft (468 kN/m)

Location of resultant:
(371,575 lbf-ft/ft) - (62,867 lbf-ft/ft)/(32,061 lbf/ft) = 9.6 ft (2.92 m)

e = (9.6 ft) - (17.25 ft/2) = 0.97 ft (0.3 m)

M_y = (32,061 lbf/ft) x (0.97 ft) = 31,099 lbf-ft/ft (140 kN-m/m)

x = 17.25 ft/2 = 8.625 ft (2.6 m)

I_y = (1 ft) x $(17.25\ ft)^3/12$ = 428 ft^4 (3.7 m^4)

$$\sigma_{\max} = \frac{32{,}061\ lbf/ft}{17.25\ ft^2} + \frac{31{,}099\ lbf\text{-}ft/ft \times 8.625\ ft}{428\ ft^4} = 2{,}485\ lbf/ft^2\ (116\ kN/m^2) \quad \textbf{(108)}$$

$(F.S.)_{\text{bearing capacity}}$ = Soil's ultimate capacity/actual contact base pressure

$(F.S.)_{\text{bearing capacity}} = (13{,}000\ \text{lbf/ft}^2)/(2{,}485\ \text{lbf/ft}^2) = 5.24 > 3.0$ OK

CHAPTER 6. INSPECTION

INTRODUCTION

This chapter describes procedures for inspection of timber substructural systems. The chapter contains sections on timber piles, steel H-piles, timber culverts, and timber cribs. The material presented is intended to provide highlights of inspection procedures and guidelines given in more exhaustive publications dealing specifically with inspection of timber bridges and timber substructures. This chapter is written based on the following publications: (1) *Bridge Inspectors Training Manual 90*, (2) *Quality Assurance and Inspection Manual for Timber Bridges*, and (3) *Bridge Inspection Manual 1990* (WV Department of Highways).[22,31,32]

SUBSTRUCTURE INSPECTION

The inspection of timber substructure elements is similar to that of substructures constructed of any other material. Essentially, there are four types of abutment movements: (1) lateral movement, (2) vertical movement, (3) settlement, and (4) rotation. These movements can be caused by one of the following: slope failure, consolidation, seepage, water table variation, frost action, drag forces, and scour. Inspection procedures for substructure stability are discussed in *Bridge Inspectors Training Manual 90*. In this chapter, inspection procedures to evaluate the condition of timber components is discussed.

Inspection of Timber Piles

This section discusses the inspection and quality control of round timber piles. Relevant ASTM and AWPA specifications are presented for preservative treatment of timber piles. For more information on inspection of timber piles, see references 4, 8, 22, and 34.

Timber Piling

All timber piling should comply with ASTM standard D25-91. Piling should be supplied in accordance with ASTM D25: table 1 is used to specify butt circumference with corresponding minimum tip circumference, and table 2 is used to specify tip circumference with corresponding minimum butt circumference. All piling should be branded in accordance with AWPA standard M-6. Handling, storage, and field treatment of the piles should conform to AWPA standard M-4.[9]

Pressure Treatment

All piling should be pressure-treated with coal-tar creosote conforming to AWPA P-1 or P-13. AWPA standard C3-91 should be used to inspect the creosote retention levels in timber piles.

Pile Driving

Timber piles should be fresh cut on the butt-end just before driving. If the area of the pile-head is greater than that of the hammer, the pile should be chamfered to at least the depth of the sapwood of the pile. Steel anvil blocks that are placed over the head of the pile should be used when driving timber piles. Further, timber pile driving should be done by striking directly on the head of the pile without the use of cushions or blocks.

Inservice Moisture Content

Since a moisture content greater than 20 percent is necessary for decay, high moisture areas should alert the inspector for a potential decay situation. Areas around fasteners and checks should be considered areas of high decay potential. A potential area for decay exits when driven drift pins and nails are used to attach a timber cap. The moisture content of the piles should be periodically measured with an electrical-resistant type moisture meter with insulated probes.

Inspection of Timber Piles After Installation

Timber piles are often exposed to constant wetting and drying, and are therefore susceptible to decay. This condition is present at the ground surface and the level of the water table. Although the inspection of the submerged section of the pile is impractical, the area of the pile near the ground surface can be inspected for decay and scour damage. The inspection of timber piles requires moisture content reading and decay assessment. In addition, the soundness of the pile section at the ground surface and the soundness of the pile to a depth of approximately 18 in (457 mm) should be checked.[31] This can be done by tapping the pile sharply and listening for a sharp ringing. The inspector can detect internal decay by a dull, hollow sound, and take necessary remedial measures. After the area in contact with the ground surface has been inspected, the remaining visible parts should be inspected for decay and moisture content. The piles should also be checked for settlement at the pile cap with the aid of an appropriate surveying instrument.

Inspection of Steel Piles

To obtain a reasonable longevity with steel H-piles or other wide-flange sections used as H-piles, it is necessary to provide corrosion protection for steel piles and to detect and monitor the corrosion deterioration of such piles. There are various methods for corrosion protection that include: (1) hot dip galvanizing, (2) paint coatings, and (3) cathodic protection. Although it is not classified as a method of protection, the use of heavier steel sections for a given application should be considered as a means of increasing the service life of the structure. For more detailed information on corrosion processes and their causes and corrosion protection methods, see references 31 and 34.

What to Look For

The primary objective of this section is to alert the bridge inspector for potential areas of corrosive activity and general inspection guidelines. For further information on steel pile inspection, see references 22 and 35.

- Check the pile bent for presence of rust at the ground-level line. Use a chipping hammer to determine the extent of corrosion.

- Check around the pile bases for debris, which should be removed.

- Check the metal clips connecting the timber lagging to the wide-flange section.

Inspection of Timber Culverts

The inspection of timber culverts follows primarily the guidelines given in the *Culvert Inspection Manual-Supplement to The Bridge Inspector's Training Manual.* These guidelines include information on the approach roadways, the waterways, and the end treatments. For additional information on culvert inspection guidelines, see references 22 and 38.

Timber culverts are as susceptible to decay and deterioration as other timber structures. However, creosote treatment is effective in extending the service life of timber culverts. AWPA standard M2-89 should be used to inspect preservative-treated lumber before, during, and after treatment. The moisture content of treated lumber can only be measured accurately using extraction assay methods outlined in AWPA standard A6-89.[36] Creosote-treated lumber will exude excessive creosote after treatment, which may be harmful. To avoid bleeding, AWPA standard C2-89 provides guidelines for acceptable retention levels and steaming after treatment. Galvanized steel dowels and other fasteners that are visible should be checked for corrosion, and the area of the timber in contact with fasteners should be checked for decay.

Inspection of Timber Cribs

Timber crib walls are subject to the same type of deterioration and decay as other timber substructures. Therefore, to ensure the longevity of a timber crib, the following items should be checked regularly following the guidelines given for inspection of culverts. For further information on inspection of timber crib walls, see references 1, 22, 31, and 39.

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*U.S. G.P.O.:1994-301-717:80512

Technical Report Documentation Page

1. Report No.	2.	3. Recipient's Catalog No.
FHWA-RD-93-024	PB95-170262	

4. Title and Subtitle	5. Report Date
TIMBER SUBSTRUCTURES FOR BRIDGE APPLICATIONS	March 1994
	6. Performing Organization Code CFC-WVU
7. Author(s) J. F. Davalos and S. H. Petro	8. Performing Organization Report No. CFC-92-148
9. Performing Organization Name and Address Constructed Facilities Center West Virginia University Department of Civil Engineering Morgantown, WV 26506-6101	10. Work Unit No. (TRAIS) 3d1a 11. Contract or Grant No. DTFH61-90-C-00067
12. Sponsoring Agency Name and Address Office of Engineering and Highway Operations R&D Federal Highway Administration 6300 Georgetown Pike McLean, VA 22101-2296	13. Type of Report and Period Covered Final Report Dec. 1991-Nov. 1992 14. Sponsoring Agency Code

15. Supplementary Notes

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16. Abstract

Timber Bridges have become a viable alternative for new bridge construction on low-volume roads, where it is imperative that the bridges be economical and long-lasting. Considerable research on superstructural systems has been completed in the U.S. and has provided design, construction, and inspection guidelines for innovative timber bridges. Guidelines for the design of stress-laminated timber decks have been published by AASHTO. However, practical recommendations concerning timber substructural systems are not readily available. Therefore, the objectives of this booklet are: (1) to present background information on timber substructures, (2) to present practical design guidelines for various systems, and (3) to present sources of additional information. The following six systems were selected: timber piles, steel bent-pile abutments, culverts, crib-wall abutments, and stub abutments. This publication is part of a collection of three booklets for the study "Education and Technology Transfer," under the Timber Bridge Research Program. The other two booklets are:

FHWA-RD-92-044 CORROSION PROTECTION OF STEEL HARDWARE USED IN MODERN TIMBER BRIDGES

FHWA-RD-91-120 DESIGN, CONSTRUCTION, AND QUALITY CONTROL GUIDELINES FOR STRESS-LAMINATED TIMBER BRIDGE DECKS

17. Key Words	18. Distribution Statement
Bridges, substructure, timber, piles, lagging, inspection.	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	111	

PREFACE

TIMBER BRIDGE RESEARCH PROGRAM

Congress, in its fiscal year 1990 Appropriations Act, directed the Federal Highway Administration to cooperate with the Constructed Facilities Center in the planning and expenditure of $500,000 for low-cost bridge technology involving wood.

This program was guided by the Timber Bridge Research Council (TBRC) formed by the Federal Highway Administration (FHWA), which includes the following representatives from the Federal, State, and county governments and industry:

Federal Highway Administration

Sheila Rimal Duwadi, P.E.
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USDA Forest Service

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Transportation Research Board

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American Association of State Highway and Transportation Officials

Donald J. Flemming, P.E.

American Wood Systems

Thomas G. Williamson

The TBRC, at a joint meeting with the National Forest Products Association (NFPA), developed and prioritized a list of research topics to be conducted by the FHWA. The following three studies were conducted under this program:

Priority	Description
1.	Timber Bridge Rail Testing and Evaluation
2.	Education and Technology Transfer
3.	Design Optimization of T, Bulb-T, and Box-Stressed Timber Bridges

These topics were endorsed by the AASHTO Technical Committee on Timber Structures. This booklet is part of the second study "Education and Technology Transfer." It provides design and inspection guidelines for timber substructural systems. Other booklets under this second study are on the design, construction, and quality control of stress-laminated timber bridge decks and the corrosion protection of steel hardware used on timber bridges.