ALLOWABLE STRESSES IN PILES

Research, Development, and Technology

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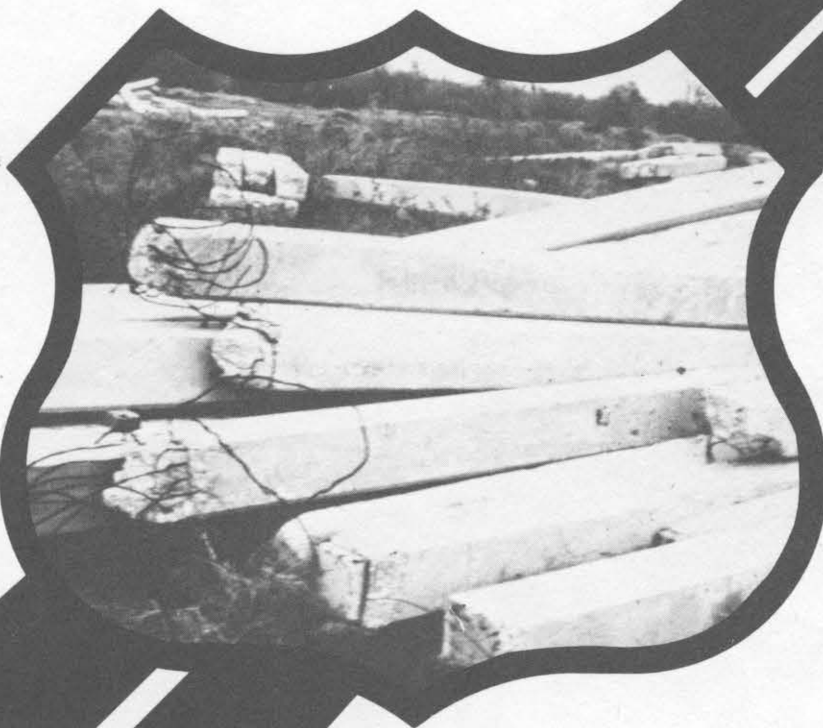

FOREWORD

This report describes new methods for establishing allowable stresses in foundation piles and will be of interest to engineers concerned with bridge and other structural foundations.

This report presents the results of Teng & Associates, Inc., research project, "Allowable Stresses in Piles." The program was conducted for the Federal Highway Administration, Office of Engineering and Highway Operations, Research and Development, Washington, D.C., under contract DTFH61-80-C-00114. This final report covers the period of research and development from October 1, 1980, to September 30, 1982.

This report presents an excellent dissertation on the subject of allowable stresses in steel, concrete and timber piles. An assessment of existing AASHTO specifications for and methods of establishing allowable stresses in piles was made, and appropriate recommendations for improving these specifications and methods are presented in the report.

Sufficient copies of the report are being distributed by FHWA Bulletin to provide a minimum of two copies to each FHWA regional office, two copies to each FHWA division office, and three copies to each State highway agency. Direct distribution is being made to the division offices.

Charles F. Galambos /for

Richard E. Hay, Director
Office of Engineering
and Highway Operations
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NOTICE

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16. Abstract

This study presents methods for establishing allowable stresses in steel, concrete, and timber piles using load/resistance factor concepts. These methods take into account not only the material properties of the pile itself but also the individual effects of long term loads, driving stresses and drivability, imperfections in form or material and various environmental conditions which tend to reduce pile capacity. Using the results of the study, changes in Section 4, Division 1 of the AASHTO Standard Specifications for Highway Bridges, are proposed.

The study is limited to the design of the pile as a structural member.

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PREFACE

This report presents results of the study to establish allowable stresses for piles used in bridge foundations. The subject study comes under Task 4, “Improved Design for Pile Foundations,” of FCP Project 4H, “Improved Foundations for Highway Bridges.”

Project objective was to define and establish, through structural analysis and supporting field data, 8 rational guideline for determining allowable stresses for pile design codes used in highway bridges. The subject research only is concerned with the pile element itself and does not involve the load-transfer or group-action aspects of pile design.

Methods developed for establishing allowable stresses were based on load factor/resistance factor design concepts. In addition to the effects of static-load conditions and material properties of the pile, the methods take into account the effects of driving stress, drivability, load eccentricity, and variation in material(s) and dimensions of the pile, 88 well as various environment81 factors (deterioration, damage due to driving) which tend to reduce pile capacity. Using these study results, changes are proposed to the AASHTO Standard Specifications for Highway Bridges.

Grateful acknowledgement is made to Professors J.E. Stallmeyer and W.L. Gamble whose comments and suggestions were particularly helpful in preparation of Chapters 4 and 5; Dr. D.M. Rempe and Mr. F.M. Fuller for their contributions to Chapter 2; numerous bridge and staff engineers from the Departments of Transportation of California, Florida, Illinois, Louisiana, Massachusetts, Nevada, New York, Pennsylvania, Texas, and Virginia for valuable information concerning individual state practices in design and construction of pile foundations;

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NOTATION

a	=	Computed 5% exclusion value of the combined frequency distribution of the combination,
A_c	=	Cross-sectional area of concrete
A_s	=	Cross-sectional area of steel
A_{sr}	=	Cross-sectional area of added steel reinforcement
A_{ss}	=	Cross-sectional area of added steel shape
β	=	Adjustment factor for load duration
b	=	Width of narrow face
b'	=	Width of the unsupported Flange (AASHTO)
b_f	=	Flange width (AISC)
c	=	Coefficient dependent on the strength property being considered
CDF	=	Composite distibution factor
D	=	Dead load or pile diameter
ϵ	=	Adjustment factor for minimum design eccentricity
ecc	=	Eccentricity factor
f	=	Adjustment factor for size and shape on modulus of rupture
f_a	=	Allowable axial stress
f_b	=	Flexural stress
F_b'	=	Allowable design stress in bending if flexural stress only existed
f_c	=	Stress in compression parallel to grain induced by axial load
F_c	=	Strength in compression parallel to grain
F_c'	=	Allowable design stress in compression parallel to grain
f'_c	=	Cylinder strength (28 day)
f_{cc}	=	Confined concrete strength
f_{ce}	=	Effective prestress in concrete
F_n	=	Strength at an inclination θ with respect to the direction of. grain
F_p	=	Strength in compression perpendicular to grain
f_s	=	Adjustment factor of safety
F_{sd}	=	Adjustment factor accounting for the composite influences of load duration and a near minimum factor of safety.
f_y	=	Specified yield strength of reinforcement
F_y	=	Specified yield strength of steel shape or pipe.
f_{ys}	=	Concentrically-loaded concrete--filled shell yield strength
γ	=	Adjustment factor accounting for the variation in strengths from butt to tip.
$\gamma_1, \gamma_2, \gamma_m$, γ_p, γ_f	=	Partial safety factors
h	=	Width of wide face
HDF	=	Hidden defect factor
I	=	Impact
k	=	Knot diameter
k_L	=	Largest knot diameter
k_S	=	Sum of knot diameters in 1 ft. (30.48 cm) pile length
L	=	Live load
LF	=	Load factor or safety factor .
m	=	Adjustment factor for in-use moisture content
M	=	Factored moment load or moisture! content at which the property P is to be determined
M_e	=	Moment at zero axial load with the outer fibers at the elastic limit,

NOTATION (continued)

M_o	=	The allowable bending moment at zero axial load (Eq. 32)
M_p	=	Moisture content at which strength etatre to change
M_p	=	Fully plastic moment with zero axial load
M_u	=	Nominal maximum moment capacity
p	=	Lateral stress
P	=	Property value at moisture content M
P_{12}	=	Property value at 12% moisture
P_a	=	Allowable load on concrete pile
P_a	=	Allowable axial load for .05D eccentricity (Eq. 33)
P_{all}	=	Allowable load for steel or concrete pile
P_g	=	Property value in green condition
P_{max}	=	Maximum axial load after reducing Po for eccentricity
P_{nx}	=	Nominal axial load strength corresponding to eccentiricty $e=e_x$
P_{ny}	=	Nominal axial load strength corresponding to eccentricity $e=e_y$
P_{nxy}	=	Nominal ultimate axial load with biaxial bending
P_0	=	Nominal concentric ultimate axial load capacity of short column
P_o	=	Allowable concentric axial load (Eq. 31)
P_y	=	Yield load in compression for short steel column
ϕ	=	Strength reduction factor or strength ratio for imperfections in timber
ϕ_b	=	Strength ratio associated with imperfections in bending
ϕ_c	=	Strength ratio associated with imperfections in compression
ψ	=	Adjustment factor modifying the property to the in-use temperature
Q	=	Service load
R	=	Ratio of the strength perpendicular to the grain to the strength parallel to grain
s	=	Average value for the property of the species
S	=	Section modulus
s'	=	5% exclusion value for the property
s_a	=	Allowable design stress
s_{ab}	=	Allowable bending design stress
s_{ac}	=	Allowable compressive design stress
s_{ad}	=	Allowable design property
s_{ag}	=	Allowable property in green lumber used as a structural member under normal load duration and normal temperature conditions
s_b'	=	Green small clear modulus of rupture
s_c'	=	Green small clear crushing strength parallel to grain
s_d	=	Standard deviation
t	=	Flange thickness (AASHTO) or pile shell thickness
θ	=	Slope of grain
t_f	=	Flange thickness (AISC)
V.I.	=	Variability index for the property of the species.
w	=	Check width
W	=	Twice the slope of grain
Z	=	Plastic section modulus

CHAPTER ONE
INTRODUCTION

The Federal Highway Administration sponsored the research described herein through their Off ice of Research under a program titled "Improved Design for Pile Foundations." The objective of the research was: to define and establish, through structural analysis and supporting field data, a rational guideline for determining allowable stress levels for pile design codes used in highway bridge foundations. Allowable pile stresses derived herein apply at any point along the embedded portion of the pile and are independent of whether the pile is a point bearing or a friction pile. The pile design load at any given point along the embedded portion of the pile is a 'function of the soil support conditions (positive or negative skin friction) assumed by the designer.

The concept of maximum allowable stress in piles has a long history. In general, the use of the concept paralleled normal structural design practice based on allowable stresses. However, the allowable stresses for various pile materials did not necessarily correlate with those allowed for the same materials when used above the foundation. The allowable stress in a pile can be used in design as follows:

1. Determine the actual load or service load likely to be borne by the pile or piles.
2. Divide by the allowable stress to arrive at the required cross-sectional area of the pile.
3. Select or proportion the pile so that it has the required cross-sectional area as a minimum.

Normally the embedded portions of piles are designed as hort columns J because even the weakest of soils surrounding piles of normal dimensions usually provides sufficient lateral support to prevent buckling.

An implied factor of safety can be determined from the material properties and the allowable stress. For example, with an A 36 steel having a yield point of 36,000 psi (248 MPa) and an allowable stress of 9,000 psi (62 MPa), a factor of safety of 4 is implied. Similarly, if the allowable stress were 12,000 psi (82.7 MPa), a factor of safety of 3 is implied. Gross implied factors of safety of approximately 3 are common for steel and concrete piles. However, such fac tors appear high compared to safety factors of 1.7 to 2.5 normally stated or implied in structural design of superstructures. Questions have been raised by steel, concrete and timber promoting agencies, and by a few engineers, as to why piles cannot be designed with allowable stresses consistent with those used in normal structural practice. This report addresses those questions and presents a rationale for prescribing allowable stresses for piles used in highway structures.

The basis for allowable stresses in piles is not usually given in building codes, whereas for ordinary structural design governed by codes, explanations of the codes are readily found in commentaries and design texts. In the three main structural codes, the concrete code (ACI 318-77) specifically excludes piles, the steel code (AISC-78) does not mention piles, and the timber code (NDS-77) covers piles on the basis of ASTM D2899-74 for which no explanation is published. Building codes are needed to insure satisfactory design, but without explanation of the reasoning behind the specified allowable stresses, the codes leave a lot to be desired. This report summarizes the -available unpublished background for current codes and practices.

Designated allowable stresses for piles vary significantly among the various building codes and from jurisdiction to jurisdiction. The highway industry is generally more conservative than the building codes of most U.S. cities and other public works agencies. AASHTO has strongly resisted efforts from private industry and others to increase their specification limits to higher values because of the concern that higher code limits would increase the incidence of construction delays caused by failed load tests, contractor claims and lawsuits, and more frequent structural failures. Higher code limits would reduce the factor of safety and increase the importance of thorough design and analysis of pile load capacity. Because the general trend outside the highway field, in the United States, has been to use higher allowable stresses in piles, there is a need to examine the consequences of increasing the designated allowable stresses.

Allowable stresses for each material type are usually established for static load conditions. Recent research results have shown, however, that the most severe stress conditions occur in the pile during driving operations. Some engineers reduce the allowable stresses of certain pile materials to account for pile deterioration caused by environmental attack (corrosion or weathering) and/or to compensate for lack of lateral support.

Because driving stresses are temporary and occur prior to the application of the superstructure loads, the consequences of over-stressing the piles during driving are generally less significant than static overloading which occurs during the design life of the bridge structure. Stress analysis during driving and pile damage inspection activities prior to superstructure loading are two methods of monitoring the condition of piles that are designed for higher allowable stress levels.

The selection of appropriate allowable stresses should first consider that the major factor in the life and performance of the pile is stability under long term applied static force; however, a dynamic analysis must also be made to insure that driving stresses are not excessive and environmental circumstances do not weaken the piles excessively. These considerations should be incorporated in a rational procedural guideline to replace currently used general stress codes. Such a guideline should have provisions for evaluating the various stress conditions separately to include separate safety margins for each condition commensurate with the probability and consequences of failure.

The project study included a search of the literature, a study of case histories, personal interviews, visits to jobsites and research stations, and a study of various codes and practices of Federal; State, and local governments. The information gathered was analyzed for practical worth to the highway industry and summarized herein for use in recommendations for improved guidelines for allowable stresses in piles. Although the study does not involve laboratory and field testing programs, the work included the development of appropriate laboratory and field investigation workplans for conducting the necessary research to establish rational guidelines for allowable stresses in piles.

Current code requirements on allowable stresses are given in Chapter Two. The AASHTO epecif ications, the practices of 10 representative States and the Canadian Bridge code are covered, Also covered are 4 model building codes, 5 representative city codes, 5 technical society recommendations, 6 trade association recommendations and 8 foreign codes.

In Chapter Three an introduction is provided to the items that should be considered in arriving at allowable stresses for pile materials. This prepares the reader for the detail analyses that follow. Detailed structural analyses of steel, concrete and timber piles are given in Chapters Four through Six, respectively. Environmental factors and other limiting factors such as corrosion, structural damage, and driving limitations appear in Chapter Seven.

A procedure for determining conditions under which higher stresses than derived herein may be used is given in Chapter Eight. Chapter Nine draws together all the recommendations for allowable stresses. The report concludes with recommendations for changes to the current AASHTO (1977) Bridge Specification (Chapter Ten).

CHAPTER TWO
CURRENT CODE REQUIREMENTS

The allowable stress requirements of over 40 agencies have been surveyed for the purpose of delineating current practice. Most agencies deal with piles by the service load design method for which allowable stresses are appropriate. A few agencies refer the designer to standard codes for structural design of timber, concrete and steel which do not contain specific provisions for piling. Two agencies operate on the basis of load factor design, but for one of them the provisions are new and full explanations of several of their provisions are not yet available.

Several codes have idiosyncrasies which make direct comparisons difficult. The approach that has been taken herein is to compare allowable stresses for steel, concrete and timber piles in separate groups. The allowable stresses presented are those that generally apply; special cases where different stresses apply are noted only when considered important.

STEEL

A list of allowable stresses for steel piles permitted by various agencies is given in Table 1. The AASHTO allowable stress of 9000 psi (62.1 MPa) has been in effect since 1965. However, the specification allows a stress of 0.5Fy baaed on load tests. Of ten States interviewed, nine generally followed the AASHTO specification; the tenth, California, allows a stress of 10,000 psi (69MPa). Two other codes developed for highway bridges are presented for comparison. The Canadian Standards Association (1978) in its "Design of Highway Bridges" allows 12,000 psi (82.5MPa). The Ontario Highway Bridge Design Code (1979) provides ultimate limit states design for foundations; thus, the generally effective allowable stress is an interpretation based on an average load factor. It appears that 11,500 psi (79.3MPa) is the approximate equivalent allowable stress. Based on high quality load testing this stress can be raised as high as 17,280 psi (119.2MPa).

Four model building codes are widely used in the United States. The National Building Code does not state an allowable stress for H-Piles, but allows 0.5Fy on steel pipe. The Basic Building Code (BOCA) allows 0.35Fy routinely, and 0.5Fy based on a load testing program. The Standard [Southern) Building Code allows 12,600 psi (86.8MPa) routinely and 0.5Fy if based on load tests. The Uniform Building Code also allows 12,600 psi (86.9MPa) routinely, but accepts 18,000 psi (124.1MPa) when proven by load test.

Four U.S. government agencies were also surveyed. The Corps of Engineers (Army) allows 10,000 psi (69MPa), whereas the Navy allows 12,000 psi (82.5MPa). The General Services Administration suggests a range from 9000-12,000 psi (62.1-82.5MPa), whereas the U.S. Post Office follows local codes.

Five major cities were surveyed. Chicago allows 12,000 psi (82.5MPa) as does Los Angeles; New York City allows 12,600 psi (86.9MPa). New Orleans allows 0.5Fy and requires load testing, but limits the allowable stress to 25,000 psi (172.4MPa). Miami (South Florida Code) allows 0.25Fy routinely, but requires a 1/16 in. thickness allowance on each face for corrosion.

The recommendations of four technical societies were also surveyed. The American Concrete Institute (ACI) does not provide a recommendation for H-piles, but recommends 12,250 psi (84.5MPa) for pipe. The American Railway Engineering Association (AREA) allows 12,600 psi (86.9MPa). The American Society of Civil Engineers (ASCE) has no current recommendations for allowable stresses in piles. Their previous recommendations were reflected in the American National Standards Institute (ANSI) Standard A56.1-52, which was withdrawn on September 7, 1976. Their steel pile recommendation was 9000 psi (82.5MPa). A task group within ASCE's Foundation and Excavation Standards Committee is currently developing new recommendations for allowable stresses.

The trade association promoting the steel producers interests in the United States is the American Iron and Steel Institute. Voting membership is strictly limited to iron and steel producers; their recommendation is 0.5Fy.

Seven foreign building codes were surveyed with mixed success in obtaining definitive allowable stresses. Several of the codes suggest that normal structural standards be used for pile design, which in turn are load. factor/resistance factor standards. Therefore, considerable interpretation was required to arrive at the values in Table 1. For the most part allowable stresses in steel are approximately 0.3Fy with Fy implied as 36,000 psi (248MPa). Several standards would allow 0.5Fy based on load tests, or if they are jacked piles (such as underpinning piles). The Danish Standard could even be interpreted to allow 0.67Fy. However, the conclusion that can be reached about foreign codes is that their practices, may be different, but the resulting allowable stresses are essentially in the same range as those used domestically.

CONCRETE

Table 2 is a list of allowable stresses for precast and cast-in-place piles that are permitted by various agencies. AASHTO allows 0.33f'c on the tip of point bearing precast piles, and 0.4f'c on concrete filled piles. It appears that 0.4f'c applies to the gross area of concrete and shell; regardless of the thickness of the shell. The ten states that were surveyed generally follow AASHTO except for the maximum design load permitted when subsurface investigation or test loading is not performed. The Canadian Standard Association Bridge Code allows 0.33f'c, whereas the Ontario Bridge Code (load factor/resistance factor) would allow values varying from 0.25f'c to 0.34f'c.

The four model codes (BOCA, NBC, SBC, UBC) uniformly allow 0.33f'c. Of the four government agencies the Army does not state a value, GSA allows 0.225f'c, the Navy allows 0.33f'c, and the Post Office follows local codes. Of the five city codes surveyed Chicago is highest at 0.4f'c and Los Angeles the lowest at 0.225f'c. New York City and Miami allow 0.25f'c, whereas New Orleans allows 0.33f'c. Of the technical societies PCA and ACI allow 0.33f'c. ASCE's last published recommendation (ANSI A56.1-52) was 0.225f'c; however, a task group in the Foundation and Excavation Standards Committee is currently preparing new recommendations for consideration by ASCE. The American Railway Engineering Association allows 0.3f'c. The foreign codes similarly vary from 0.22f'c to 0.33f'c.

TIMBER

Allowable stresses on timber piles vary widely partly because of the many species, each having its own characteristic range of strength, and

TABLE 1 - ALLOWABLB STRESSES ON STEEL PILES

AGENCY/CODE	ALLOWABLE STRESS PSI (MPa)	REMARKS
AASHTO[1]	9,000 (62.1)	0.5Fy if based on tests
California	10,000 (69.0)[2]	
Florida	Same as AASHTO[2,3]	
Illinois	Same as AASHTO[2]	
Louisiana	Same as AASHTO[2]	
Xaeeachusetts	Same as AASHTO[2]	80 tons (712 kN) max on pipe piles
Nevada	Same as AASHTO[2]	Follow Cal. D.O.T. specs.
New York	Same as AASHTO[2]	Use reinforced tips
Pennsylvania	Up to 14000 (96.6)[2]	
Texas	Same as AASHTO[2,3]	
Virginia	Same as AASHTO[2]	
Canadian Std. Assn.	12,000 (82.5)	
Ontario Bridge Code	11,500 (79.3)	Load test req'd. above 11,500 (79.3)
Basic Building Code	0.35Fy	0.5Fy if based on tests
Nat. Bldg. Code	0.5Fy pipe	H-unstated
Standard Bldg. Code	12,600 (86.9)	0.5Fy if based on tests
Uniform Bldg. Code	12,600 (86.9)	18,000 (124.1) if based on tests
U.S. Army	10,000 (69.0)	
U.S. GSA	9-12,000 (62.1-82.5)	
U.S. Navy	12,000 (82.5)	
U.S. Post Office	Follow local codes	
Chicago	12,000 (82.5)	
Los Angeles	12,000 (82.5)	0.5Fy if based on tests
New Orleans	0.5Fy Test req'd.	Fy=50 ksi (345 MPa) max.
New York City	12,600 (86.9)	
South Florida (Miami)	0.25Fy	0.5Fy if based on tests
ACI 543R	12,250 (84.5)	Pipe, maximum
ANSI A56.1-52	9,000 (82.5)	
AREA	12,600 (86.9)	
ASCE[4]	--	No current recommendations
AISI, 1973	0.5Fy	
Australia	0.4Fy	0.5Fy if based on tests
Canada	0.3Fy	0.5Fy if based on tests
Denmark	0.33-0.67Fy	Upper limit based on test
England	0.3Fy	0.5Fy if jacked
Germany	0.5Fy max.	
Japan	0.3-0.4Fy ±	2mm corrosion deduction
New Zealand	0.3-0.4Fy ±	1/16" (1.6mm) corrosion deduction
Sweden	8700-12,325 (60-85)	

1 For pipe piles MSHTO allows 0.4 f'c over gross area of concrete and steel.
2 Follow AASHTO 1.4.4(B) for determining capacity of pile as structural member.
3 Maximum design load-for point bearing piles may be different from AASHTO 1.4.4(E).
4 Task Group recommendations to full Foundation and Excavation Standards Committee are not yet available.

TABLE 2 - ALLOWABLE STRESSES ON CONCRETE PILES'

AGENCY/CODE	ALLOWABLE STRESS PSI (MPa)	REMARKS
MSHTO	0.33 f'c	0.4 f'c on cast-in-place
California	AASHTO[1]	45-70 tons (400-623 kN)
Florida	AASHTO[1,2]	
Illinois	AASHTO[1]	45 tons (400 kN) on 12 in. (305 mm.) metal shell
Louisiana	AASHTO[1,2]	
Massachusetts	AASHTO[1]	
Nevada	AASHTO[1,2]	Follow Cal. D.O.T. specs.
New York	AASHTO[1,2]	12 in.(305 mm.)C.I.P. to 50 tons (445 kN)
Pennsylvania	AASHTO[1,2]	
Texas	AASHTO[1,2]	Test to establish capacity
Virginia	AASHTO[1,2]	30-70 tons (267-623 kN)
Canadian Std. Assn.	0.33 f'c	
Ontario Bridge Code	0.25-0.34 f'c	
Basic Building Code	0.33 f'c	
Nat. Bldg. Code	0.33 f'c	
Standard Bldg. Code	0.33 f'c	
Uniform Bldg. Code	0.33 f'c	
U.S. Army		Not Stated
U.S. GSA	0,225 f'c	
U.S. Navy	0.33 f'c	
U.S. Post Office	Follow local codes	
Chicago	0.4 f'c	Cast-in-Place
Los Angeles	0.225f'c	
New Orleans	0.33 f'c	0.25 f'c cast-in-place
New York City	0.25 f'c	
South Florida (Miami)	0.25 f'c	
ACI 543R	0.33 f'c + reinforcing	
ANSI A56.1-52	0.225 f'c	
AREA	0.3 f'c	
ASCE[4]	--	No current recommendations
PCA	0.33 f'c	
Australia	0.3 f'c	0.25 f'c uncased
Canada	0.33 f'c ±	
Denmark	0.33 f'c	
England	0.3 f'c ±	
Germany	0.33 f'c	
Japan	0.25 f'c	
New Zealand	0.225 f'c	
Sweden	943-1088 (6.5-7.5)	

1 Follow AASHTO 1.4.4 (B) for determining capacity of piles as structural member.
2 Maximum design load for point bearing pile may be different from AASHTO 1.4.4 (E).
3 f'c is 28 day strength of concrete.
4 Task Group recommendations to full Foundation and Excavation Standards Committee are not yet available.

partly because of different views on factor of safety. For comparison, the study was limited to the three most common species used for piles in the United States, namely, Douglas fir, southern pine, and red oak. Table 3 is a comparison of allowable stresses permitted by various agencies.

AASHTO alloys 1200 psi (8.3MPa) on Douglas fir, 1100 psi (7.6MPa) on red oak, and 1200 psi (8.3MPa) on southern pine. California allow8 up to 45 tons on all timber piles with a butt diameter of 12 inch or greater without being specific regarding allowable stress. Nevada allows up to the range of 50-70 tons, generally on Douglas fir with 10 inch-diameter tips, but is careful to do this only in soil conditions where substantial skin friction exists, thus relieving tip loading. The Canadian Standard Association allows 1088 psi (7.5MPa), whereas the Ontario Bridge Code allows 1200 psi (8.3MPa), both on Douglas fir. No firm data were available on their treatment of other species.

Three of the four model building codes (Basic, National and Standard) follow ASTM which allows 1250 psi (8.6MPa) on Douglas fir, 1100 psi (7.6MPa) on red oak and 1200 psi (8.3MPa) on southern pine. The Uniform Building Code is the same except that the allowable stress for Douglas fir is lowered to 1200 psi (8.3MPa). ASTM mentions that no formal factor of safety has been employed in arriving at their recommended stresses. By contrast the National Forest Products Association recommends that a factor of safety of 1.25 be applied to the ASTM values.

Allowable stresses recommended by several U.S. Government agencies vary from 1000 psi (6.9MPa) for GSA to 1200 psi (8.3MPa) for the Navy. The Army does not mention a timber pile allowable stress, and the Post Office follows local practice. Of the cities surveyed, Chicago is silent on allowable stress for timber, Los Angeles allows 1000 psi (6.9MPa), New York City allows 1200 psi (8.3MPa), Miami follows NFPA recommendations, and New Orleans follows ASTM, but limits the load to 25 tons (222kN) total.

Interesting differences are noted in the recommendations of the technical societies. The highest allowable stresses are recommended by ASTM. However, ASTM Committee D07.07 is currently reviewing recommendations from a task group to revise their allowable stresses. The task group within ASCE's Foundation and Excavation Standards Committee is also preparing new recommendations to the full committee for timber pile allowable stresses. ASCE's last published recommendation (withdrawn ANSI A56.1-52) was an allowable stress of 800 psi (5.5MPa) for all species considered. AREA recommends 800 psi (5.5MPa) for point bearing piles and 1200 psi (8.3MPa) for friction piles, which is a way of recognizing that load transfer holds the tip stress of 800 psi (5.5MPa) or less on friction piles.

Foreign code recommendations on allowable stress were difficult to assess primarily because of differences in species and grading standards. However, Australia appears to allow stresses in the range of 957-1523 psi (6.6-10.5MPa). Japan allows 711 psi (4.9MPa), New Zealand 750 psi (5.3MPa) and Sweden 653 psi (4.5MPa).

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TABLE 3 - ALLOWABLE STRESSES IN TIMBER[1] PILES

AGENCY/CODE	ALLOWABLE STRESSES, PSI (MPa) Douglas Fir	Red Oak	So. Pine	REMARKS
MSHTO	1200(8.3)	1100(7.6)	1200(8.3)	
California	--	--		Up to 45 tons (400 kN)
Florida				Follow AASHTO[3,4]
Illinois				24 tons (214kN) to 50 ft. (15.2m) long[3]
Louisiana				To 40 tons (356 kN)[3,4]
Massachusetts				Follow AASHTO
Nevada				Up to 70 tons (623 kN)[3]
New York				Timber seldom used
Pennsylvania				Timber no longer used
Texas				Timber not used
Virginia				20 tons (178 kN) for all piles
Canadian Std. Assn.	1088(7.5)	--	--	
Ontario Bridge Code	1200(8.3)±	--	--	
Basic Building Code	ASTN	ASTM	ASTM	
Nat, Bldg. Code	ASTM	ASTM	ASTM	
Standard Bldg. Code	ASTM	ASTM	ASTM	
Uniform Bldg. Code	1200(8.3)	1100(7.6)	1200(8.3)	
U.S. Army	--	--	--	Not Nentioned
U.S. GSA	1000(6.9)	1000(6.9)	1000(6.9)	
U.S. Navy	1200(8.3)		1200(8.3)	
U.S. Post Office	--	--	--	Follow local Codes
Chicago	1200(8.3)	1000(6.9)	900(6.2)	Load test required for 25 ton (222kN)
Los Angeles	1000(6.9)	1000(6.9)	1000(6.9)	
New Orleans	ASTM	ASTM	ASTM	25 tons (222 kN) max.
New York City	1200(8.3)	1200(8.3)	1200(8.3)	
South Florida(Miami)	NFPA	NFPA	NFPA	
ANSI A56.1-52	800(5.5)	800(5.5)	800(5.5)	
AREA	--	--	--	800-1200(5.5-8.3)
ASCE[5]	--	--	--	No recommendations
ASTM D2899-74	1250(8.6)	1100(7.6)	1200(8.3)	No factor of safety
NFPA	1000(6.9)	800(6.1)	960(6.6)	Factor of safety=1.25
Australia'				957-1523(6.6-10.5)
Canada[2]				
Denmark[2]				
England[2]				
Germany[2]				
Japan[2]				711(4.9)
New Zealand[2]				750(5.3)
Sweden[2]				653(4.5)

1 Treated Timber
2 Different species involved. Numbers given are approximate equivalents.
3 Follow AASHTO 1.4.4 (B) for capacity of the'pile as a structural member.
4 Maximum load for point bearing pile may be different from MSHTO 1.4.4 (E).
5 Task Group recommendations to full Foundation and Excavation Standards Committee are not yet available.

AASHTO HISTORY

The first appearance of the AASHO (American Association of State Highway Officials) Standard Specifications for Highway Bridges was in July 1927, and evidently was tentative. The version appearing in 1931 is labeled Edition 1 herein because the third volume, which appeared in 1935, was labeled "second edition", To date there have been twelve editions plus the first volume,

The allowable stresses as published by AASHO/AASHTO in their various editions are given in Table 4. No specific allowable stresses for piles were mentioned until 1941, and this was done indirectly. For example, the maximum load for various sizes of H-piles was given, and corresponded very closely to an allowable stress of 5200 psi (35.9MPa). Concrete piles were limited to 25 tons (222kN) to 43 tons (400kN) which amounted to very low allowable stresses and certainly less than 0.25f'c. Timber piles were limited to the range of 18-25 tons (160-222 kN); thus, the allowable stresses listed in Table 4 were not likely to be the governing factor.

In 1944 AASHO specifically stated an allowable stress of 6000 psi (41.4MPa) for steel piles. This allowable prevailed until 1965 when it was raised to its present value of 9000 psi (62.1MPa). Specific allowable stresses for concrete piles were stated in 1949 and can be interpreted to vary from 0.25f'c to 0.40f'c. It is judged, herein that 0.33f'c applies to precast non-prestreesed piles and 0.40f'c to cast-in-place pipe or shell piles. These stresses prevailed through the 1977 edition also.

For timber piles, a minor allowable stress change from 1000 psi (6.9MPa) to 1200 psi (8.3MPa) occurred in 1933. These figures pertain to the highest stress grades allowed by AASHO/AASHTO. In practice, timber piles are usually limited to design loads that result in stresses lower than those quoted above; normal engineering practice is to limit timber pile design loads to 10 to 30 tons (89-267 kN) because the harder driving required to develop higher pile loads often results in excessive breakage.

TABLE 4 - HISTORY OF AASHTO PILE ALLOWABLE STRESSES

Edition/Year		Steel psi (MPa)	Concrete[1] psi (MPa)	Timber, psi (MPa) Douglas Fir	Red Oak	Southern Pine	Remarks[2]
0	1927	--	--	--	--	--	No specifics for piles
1	1931	--			--	--	No specifics for piles
2	1935	--	--	--	--	--	No specifics for piles
3	1941	5200+(34.9)[3]	See Remarks	1200(8.3)	1100(7.6)	1000(6.9)	Conc. 25-40 tons Timber 18-25 tons
4	1944	6000(41.4)	See Remarks	1200(8.3)	1100(7.6)	1000(6.9)	Conc. 20-50 tons Timber 18-28 tons
5	1949	6000(41.4)	0.25-0.40 f'c	1200(8.3)	1100(7.6)	1000(6.9)	
6	1953	6000(41.4)	0.25-0.40 f'c	1200(8.3)	1100(7.6)	1200(8.3)	
7	1957	6000(41.4)	0.25-0.40 f'c	1200(8.3)	1100(7.6)	1200(8.3)	
8	1961	6000(41.4)	0.25-0.40 f'c	1200(8.3)	1100(7.6)	1200(8.3)	
9	1965	9000(62.1)	0.25-0.40 f'c	1200(8.3)	1100(7.6)	1200(8.3)	
10	1969	9000(62.1)	0.25-0.40 f'c	120068.3)	1100(7.6)	1200(8.3)	
11	1973	9000(62.1)	0.25-0.40 f'c	1200(8.3)	1100(7.6)	1200(8.3)	
12	1977	9000(62.1)	0.25-0.40 f'c	1200(8.3)	1100(7.6)	1200(8.3)	

1 f'c is 28 day strength of concrete
2 1 ton = 8.896 kN
3 Maximum load vs. size equates to approximately 5200 psi (35.9 MPa)

CHAPTER THREE
FACTORS INFLUENCING PILE ALLOWABLE STRESSES

The allowable stress often is considered as an ultimate stress divided by a suitable factor of safety. In the structural design of piling as a compression member the ultimate stress would be that stress which would cause yielding or buckling failure of the pile. Experienced engineers, on the other hand, know that a more realistic view considers several factors not otherwise apparent in codes or standards e.g. the correctness of the estimate of the applied loads, the accuracy of predicting the ultimate strength of the structural element and the consequences of the failure of the element. In fact, experienced engineers may calculate the gross implied factor of safety as an afterthought solely for the purpose of comparison with other disciplines rather than using the factor of safety as a method of deriving the allowable stress.

The purpose of the following discussion is to examine:

1. Those factors that should be accounted for in arriving at an allowable stress;
2. Concepts of safety;
3. Two groups of factors: one that causes an increase in load and the other which causes a decrease in resistance; and
4. Structural considerations and the concept of pile drivability

CONCEPT OF SAFETY

Factor of Safety - If a load of one unit is applied to a structural member with an ultimate resistance (strength at failure by yielding or instability) of two units, the factor of safety is said to be two. Unfortunately, in practice neither the load nor the ultimate resistance is known with certainty, resulting in an uncertain factor of safety. Code required factors of safety are based on past experience so that the incidence of failure is either tolerable or improbable. In other words the gross factor of safety was adjusted to cover the load and structural resistance variations either empirically or by judgement, or both. Recently, engineers have devoted great effort to refining concepts of safety.

Design Using Load Factor/Resistance Factor - This procedure divides safety provisions into two parts, one dealing with loads and the other with resistance or structural strength. Reinforced concrete is commonly designed with this method using load factors applied to service loads and strength reduction factors (ϕ-factors) applied to the nominal ultimate strength for structural elements.

Load factors assess the possibility that prescribed service loads may be exceeded. Obviously, a live load is more apt to be exceeded than a dead load which is largely fixed by the weight of the construction. The ultimate strength of the members must accommodate the total of all service loads, each multiplied by its respective load factor; the load factors are different in magnitude for dead load, live load, and wind or earthquake loading, etc.

The ϕ factors are provided to allow for variations in materials, construction dimensions, and calculation approximations; all these factors are at least partially under the control of the engineer. At present U.S.

Strength reduction factor

practice considers ϕ to vary only with the type of stress or member considered, that is, shear, flexure or whether a compression member is involved.

The AASHTO (1977) load factor/resistance factor approach to reinforced concrete design is quite similar to the ACI code (ACI 318-77). For example, AASHTO specifies factors of 1.3 on the dead load (D) and 2.17 on live load plus impact (L+I). As applied to a tied column for which the ϕ factor is 0.7, the design expression becomes:

$$1.3D+2.17(L+I)= 0.7(0.85f'_cA_c+f_yA_s)$$

Where f'c is the 28-day cylinder strength, fy is the yield stress for reinforcement and Ac and As are the areas of concrete and steel respectively. It should be noted that the 0.85 factor is applied to f'c and represents a product/sample ratio, or factor. In other words, the concrete in the column (product) behaves with only 85 percent of the strength observed in a sample cylinder. The left side of the above equation deals only with loads; it could be changed to any of a number of cases of loading as is common in bridge design. The right side of the equation represents the reliable ultimate strength and does not change with the loading; it should always equal or exceed the factored loads.

If it is assumed that the service load is constant and member strength equals the nominal value, then the factor of safety is:

$$FS = \frac{\text{Load Factor}}{\phi - \text{Factor}}$$

Note that the factor of safety changes if either the load factor or ϕ-factor changes. The number arrived at for the factor of safety in the above expression probably causes more confusion than enlightenment in that it is often assumed that service loads are constant and that nominal strengths prevail. A more realistic view of the situation is presented in Figure 1. It is recognized that the service load is not a constant and has some statistical distribution. Further, member strengths are not equal to the nominal value and also have a statistical distribution. In a particular case the difference between member strength and load is called the margin. If the margin is positive, failure does not result. If the margin is negative, such as in the cross-hatched zone in Figure 1, failure results.

The load factor and the ϕ-factor are chosen such that the load and strength distributions on Figure 1 do not overlap, or overlap (cross-hatched zone) with an acceptable percentage of failures. With the foregoing as a criterion it should now be apparent that the indicated nominal factor of safety is only a by-product and that the controlling factors are really the variations in loading and strength that occur in practice.

Partial Factors of Safety - The use of partial factors of safety is essentially European in origin, and as currently applied it addresses more sources of variations in load and resistance than the Load Factor/Resistance Factor method outlined previously. The two methods are really the same, with the method of partial factors of safety being the more general case. As used, mean resistance (analagous to nominal resistance) is divided by a series of partial factors, all equal or greater than unity. The mean value of load (analagous to service load) is multiplied by a series of partial factors, all equal or greater than unity.

For example, mean resistance can be divided by three factors, as follows:

γ_m = a material partial safety factor which factors the material strength to an acceptably safe value.

γ_f = a fabrication partial safety factor which factors the resistance to account for fabrication or construction, and,

γ_p = a professional partial safety factor which factors the resistance to account for design errors or inadequate theory or bias.

Similarly, variations in load are considered by multiplying by two factors, as follows:

γ_1 = a partial safety factor which factors the load to an acceptably safe level, and

γ_2 = a partial safety factor which accounts for the possibility of (1) an increase in load due to change of use of the structures from the purpose for which it was designed; (2) the loads not being representative due to errors in design assumptions; (3) the loads being larger due to construction effects; (4) the load being larger due to temperature effects or creep; and (5) the load being in error because the assumed probability density function is incorrect.

Further expansion of the system is obtained by tabulating values for γ_2 on the basis of good, normal or poor control, which can relate to material supply, construction quality, loading information, or other pertinent items. This provides the engineer an apparently rational procedure for expressing his doubts or confidence in the resulting structure, whatever the case may be.

The procedure now used by AASHTO is essentially Load Factor/Resistance Factor design. This report refers only to that method. However, the information developed herein is in a form that facilitates a change to partial factors at some future date.

INCREASES IN LOAD

Numerous factors cause increases in loads on piles. These factors should be considered in decisions on allowable stresses and margin of safety. They include overload, negative skin friction, load transfer analysis, group behavior, pile mislocation, differential settlement, and construction activities.

Overload - Bridge engineers deal with numerous cases of loading in their designs. However, overloads may be permitted on a bridge based upon political expedience. These overloads may exceed the factored load used in design. Data on this source of load is obviously lacking, but experienced engineers recognize its existence.

Negative Skin Friction - When future consolidation of the foundation soil is anticipated by the Engineer, negative skin friction (down drag) is usually included as a pile load. However, the reason for listing this

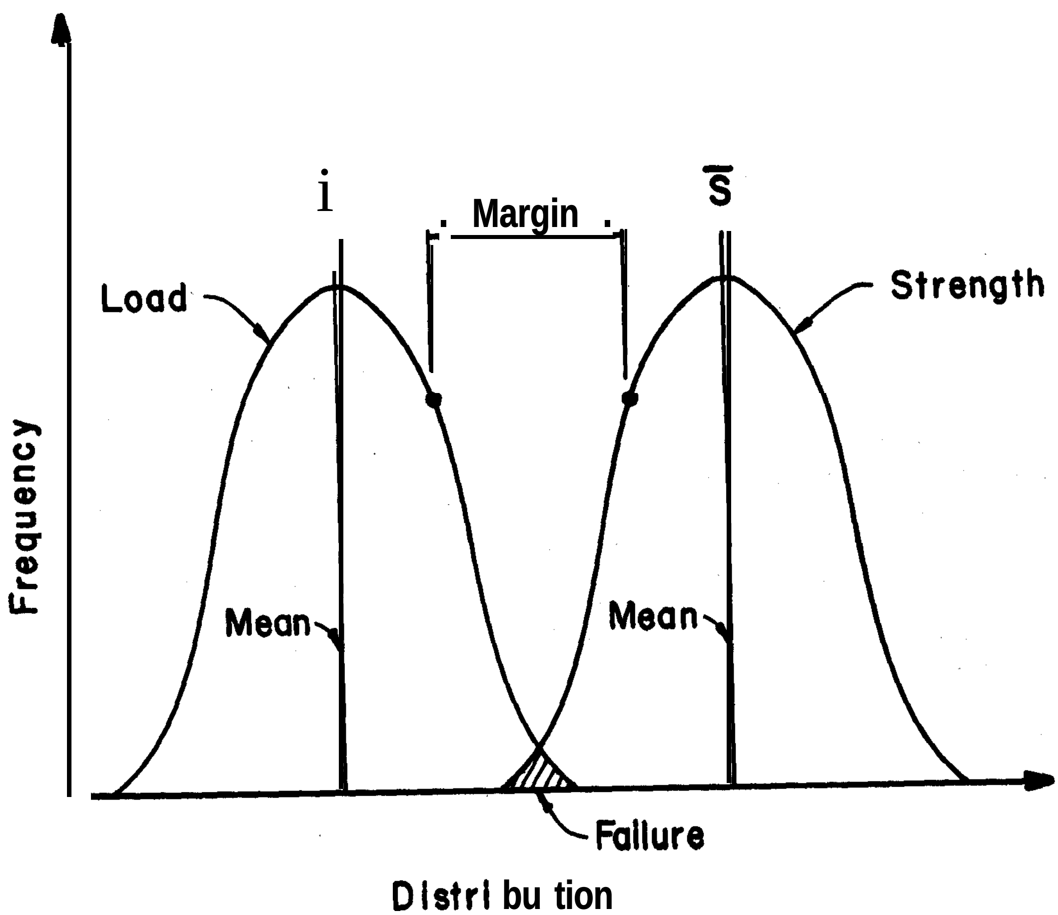


Figure 1. Load and Strength Distribution.

topic separately is that down drag may be overlooked by the designer. Further, future events in the vicinity of a completed structure that are beyond the designer's comtemplation may cause down drag to develop. Examples include the construction of a fill adjacent to a structure, and lowering the water table in a compressible area adjacent to the structure.

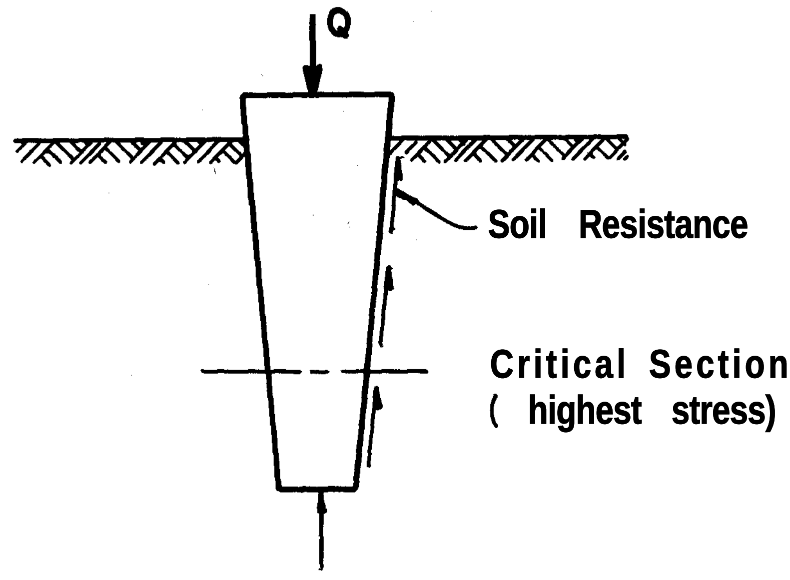
Down drag is particularly pernicious in that it alters the location of soil support for a pile. Positive skin friction that develops in a load test or normal service conditions (Figure 2a) is reversed (negative) and now acts downward, resulting in transmission of not only the pile load but also a soil load further down the pile. Down drag is quite important in the case of tapered piles (wood, monotubes, step-taper) which may have been justified on the assumption of positive skin friction in a load transfer analysis. Wood is the weakest of the tapered piles named; several examples of failure of wood piles due to down drag exist in the literature. (Carlanger and Lambe, 1973)

By comparison of Figures 2a and 2b it can be seen that a load test does not prove the structural adequacy of a pile for a given application when down drag is likely to be a factor. In the load test positive skin friction is forced to exist at all points along the pile resulting in the least amount of load transfer towards the pile tip. By contrast, under service conditions when down drag develops, load transfer towards the pile tip is greater than that observed in a load test. Thus, considerable caution is in order when designing for down drag, even when the design is aided by pile load testing.

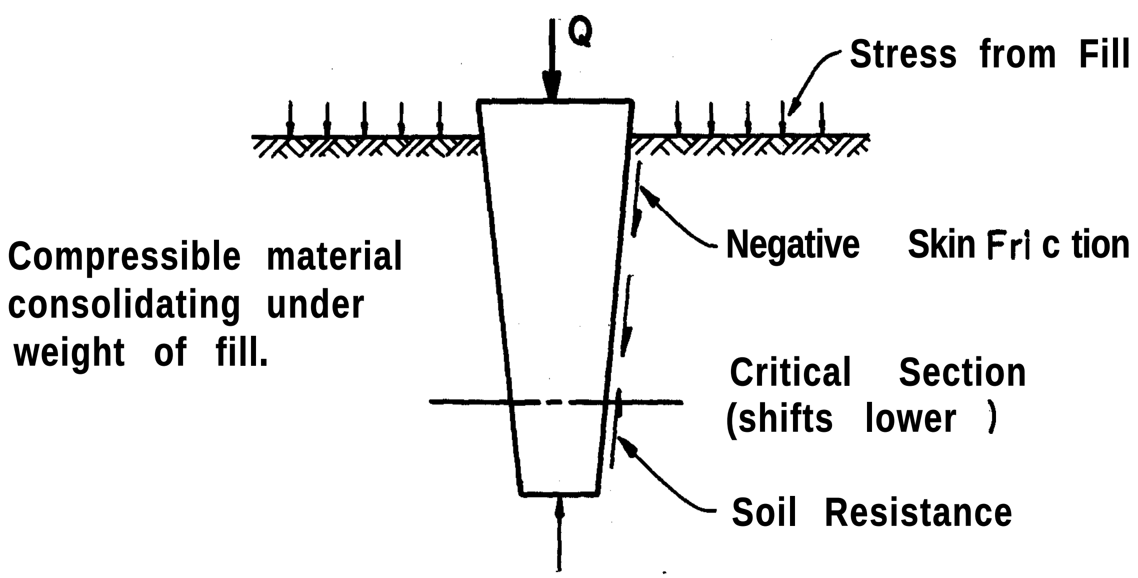
Load Transfer Analyses - Inadequate design theory or calculation errors can result in underestimating applied loads. When dealing with tapered piles, it is essential that a load transfer analysis be performed unless the pile tip has sufficient strength to resist the entire pile design load. Many methods of load transfer analysis exist, reflecting the unsettled state of the art. Room for error on the unconservative side exists, which in this case results in an increase of load on deeper portions of the pile that has a progressively smaller ability to resist load. The result is similar to lowering of the critical section as described in Figure 2b.

Croup Behavior - Both model studies and full scale field tests on groups of friction piles demonstrate that exterior piles carry higher than average loads, whereas interior piles carry lower than average loads (Whitaker, 1957; Cooke, et al, 1980; Cooke et al, 1981). Corner piles carry the highest loads with other exterior piles being intermediate. These phenomena can be understood by imagining the downward displacement of an assumed rigid pile cap under service load Q such as that in Figure 3a. The exterior piles immediately begin transferring load to the soil by skin friction resulting in a downward dish-shaped depression of the soil, Figure 3b. However, the interior piles are subjected to a lesser relative displacement of pile to soil and hence cannot develop as much skin friction (or load in the pile).

Somewhat different results are observed for a flexible pile cap, Figure 3c. In this case the piles may be uniformly loaded, but pile displacements are unequal. The corner piles settle least in accepting their share of load, whereas the interior piles settle the most; the other exterior Piles are intermediate. In practice, most groups of piles have caps of



a) Load Test and Normal Service Condltions



b) Service Condition With Negative Skin Friction

Figure 2. Pile Load Test and Negative Skin Friction (Down 'Drag)

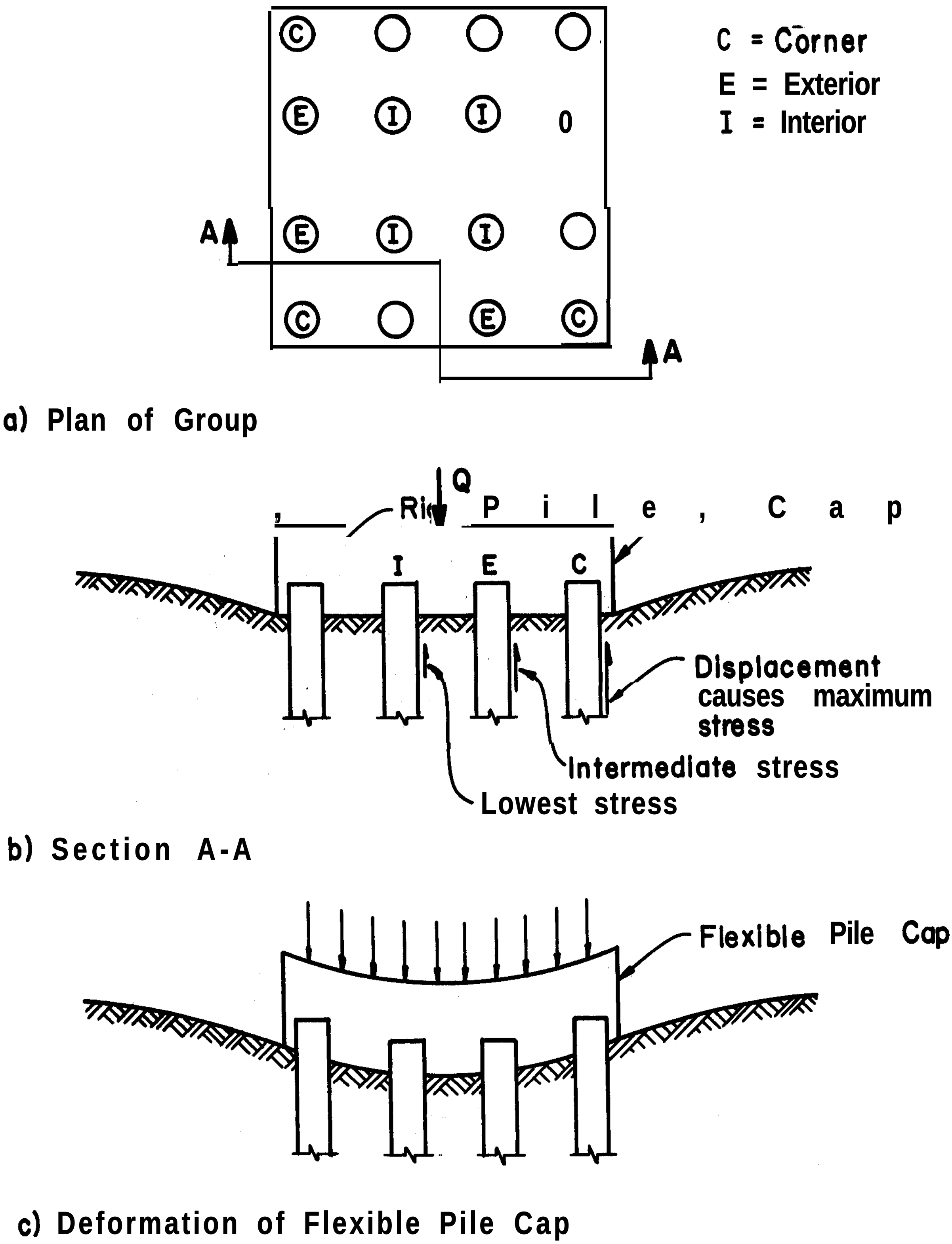


Figure 3. Behavior of a Group of Piles,.

intermediate stiffness.

The practical significance of the foregoing behavior is that piles must have the ability to accept significant overloads in normal service. The magnitude of these overloads may be in the range of 10 to 70 percent of the average loads.

Pile Mislocation - Pile construction specifications commonly allow three inches or more of pile mislocation from the design location. Pile mislocations shift the center of gravity of the group o'f piles resulting in eccentric loading, which increases the load on some piles and decreases the load on others. Small groups of say two to four piles are most susceptible to overload. For example, a 4-pile cluster spaced at 3 ft. center-to-center subjected to optimum mislocations of up to 3 inches causes the most heavily loaded pile to be loaded to 124 percent of the average load. This effect diminishes as the.number of piles in the group increases.

The AASHTO specifications wisely suggest that mislocation be considered in design. This is important to pile cap design also, especially with the methods used to determine shear in the cap.

Differential Settlement - With indeterminate structures, differential settlement of one support shifts load to other. supports; therefore, pile loads can be increased by differential settlement.

Construction Activities - Several common situations arise in construction that induce moment loads in piles. Almost all piles are driven with some curvature that locks moments into the piles*; this is most pronounced if the pile assumes a dog-leg shape. Another source of curvature is lateral displacement of previously driven piles caused by driving additional piles; this occurs primarily in soft saturated cohesive soils that are most noted for heave, and in any soil if driving advances toward an open excavation.

Another source of curvature in piles is lateral displacement caused by driving piles in a slope. Previously driven piles typically displace down-slope as additional piles are driven. Similar movements occur if a trench is cut adjacent to previously driven piles. The release of lateral earth pressures results in lateral movements toward8 the trench, thus inducing moments in the piles.

DECREASES IN RESISTANCE

Numerous factors cause decreases in the resistance offered by piles. These factors should be considered in decisions on allowable stresses and margin of safety, They include material variations, pile damage, heave, inadequate inspection, and corrosion.

Material Size and Strength Variations - Material purchase specifications usually have tolerances on size of the pile cross-section. Also, all materials exhibit a normal range in strengths. Thus, it is possible for a pile to be understrength because it is undersize or contains basic material weaknesses, or both.

Pile Damage - This is an obvious cause of a decrease in structural resistance of a pile. Unfortunately, in solid pile cross-sections, the damage usually occurs below ground, out of sight, and goes undetected. Pipe and shell piles, on the other hand, can be inspected for damage prior to concreting. Figures 4 and 5 show examples of damage to steel and timber piles respectively.

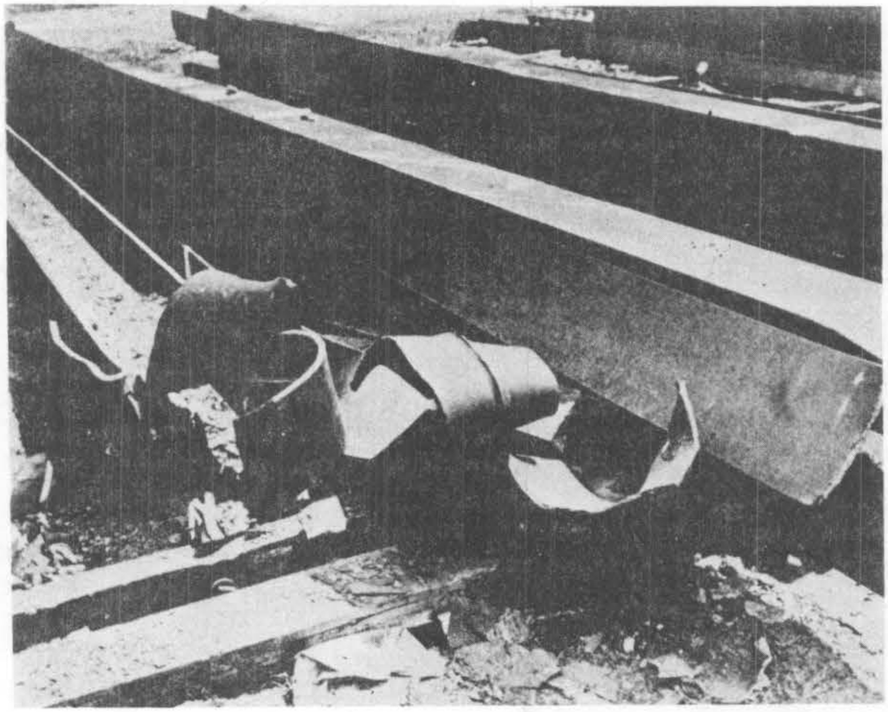


Figure 4 - Steel Pile Damage

Figure 5 - Timber Pile Damage

Another source of damage is collision by construction equipment. This can cause obvious damage at the surface, and undetected damage below ground due to flexural failure, particularly in cast-in-place piles and auger grout piles that do not contain reinforcing steel.

Heave - Soil displacements caused by driving adjacent piles can result in lateral and upward movement of the ground which in turn can exert upward forces on nearby piles. There are instances on record where unreinforced cast-in-place piles up to 40 ft. from the pile being driven have been heaved 7 inches resulting in horizontal fractures and separation in the concrete.

Inspection - Human failure in inspection can result in detectable damage being overlooked, resulting in a decrease of pile strength.

Corrosion - Corrosion of steel and deterioration of concrete and timber are obvious strength reducing factors. Where corrosion influences exist, the damage can be dramatic, as shown in Figure 6.

DRIVABILITY AND SOIL FREEZE

Drivability is loosely defined as the static soil resistance that a hammer-cushion-pile system can overcome. This equates to the static pile ultimate load capacity at the time of driving. The wave equation analysis of pile driving is currently the best method of estimating drivability (Smith 1955, 1962). Results of wave equation analyses are typically presented as plots of static ultimate load capacity vs. driving resistance (hammer blows per inch). The peak average axial stress in the pile is also usually plotted vs. driving resistance. Typical results are presented in Figure 7.

Soil freeze is a well known phenomenon that causes a pile to gain ultimate load capacity with time after driving. Many soils exhibit freeze which is usually attributed to soil reconsolidation after disturbance by pile driving. There are also many soils that do not exhibit freeze. On the other hand, some soils exhibit relaxation, which is a loss of ultimate pile load capacity with time after driving.

A measure of soil freeze or relaxation can be developed by wave equation analyses coupled with a static load test carried to ultimate resistance. For example, the wave equation analysis for a load test pile is shown in Figure 7; the pile was driven to 10 blows/inch (blows/25mm) final driving resistance and the analysis indicates 150 tons (1334kN) ultimate load capacity at the time of driving. If the load test indicates an ultimate load of 150 tons (1334kN), it is said that no soil freeze exists. If the load test indicates an ultimate load of 200 tons (1779kN), it is said that 50 tons (445kN) of freeze exists. That is, freeze is the difference between the load test result and the wave equation analysis. More rarely, if the test indicates an ultimate load of 125 tons (1112kN), it is said that 25 tons (222kN) of relaxation exists (difference between test and wave equation analysis).

The foregoing concepts have great significance in pile foundation practice. First in importance is drivability. The elements controlling drivability are under the control of the engineer and subject to design. These elements are the hammer, hammer cushion, and the pile. On the other hand, soil freeze is a phenomenon that the engineer can only observe; he cannot control it. If the engineer designs piles for ultimate loads within

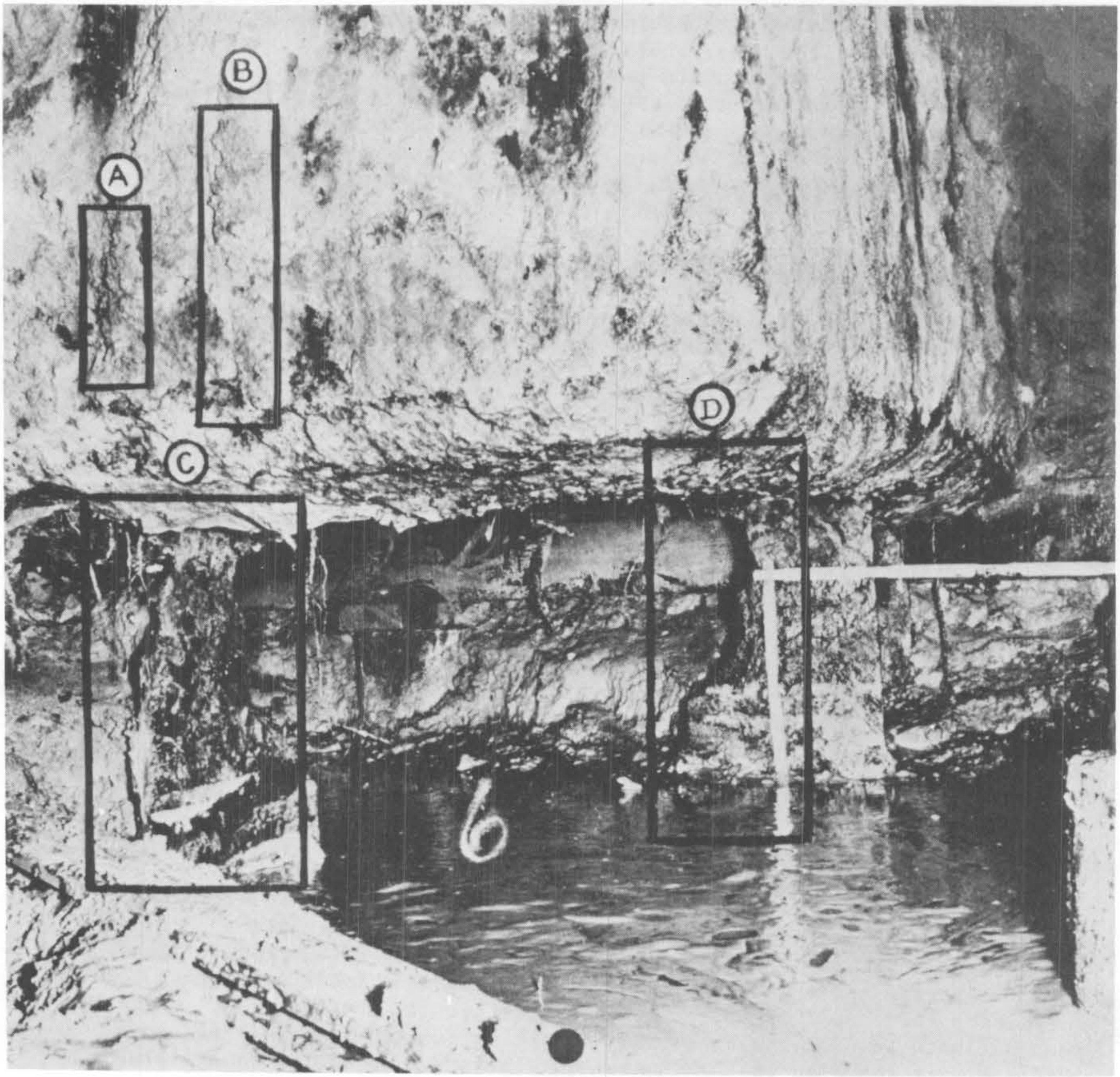
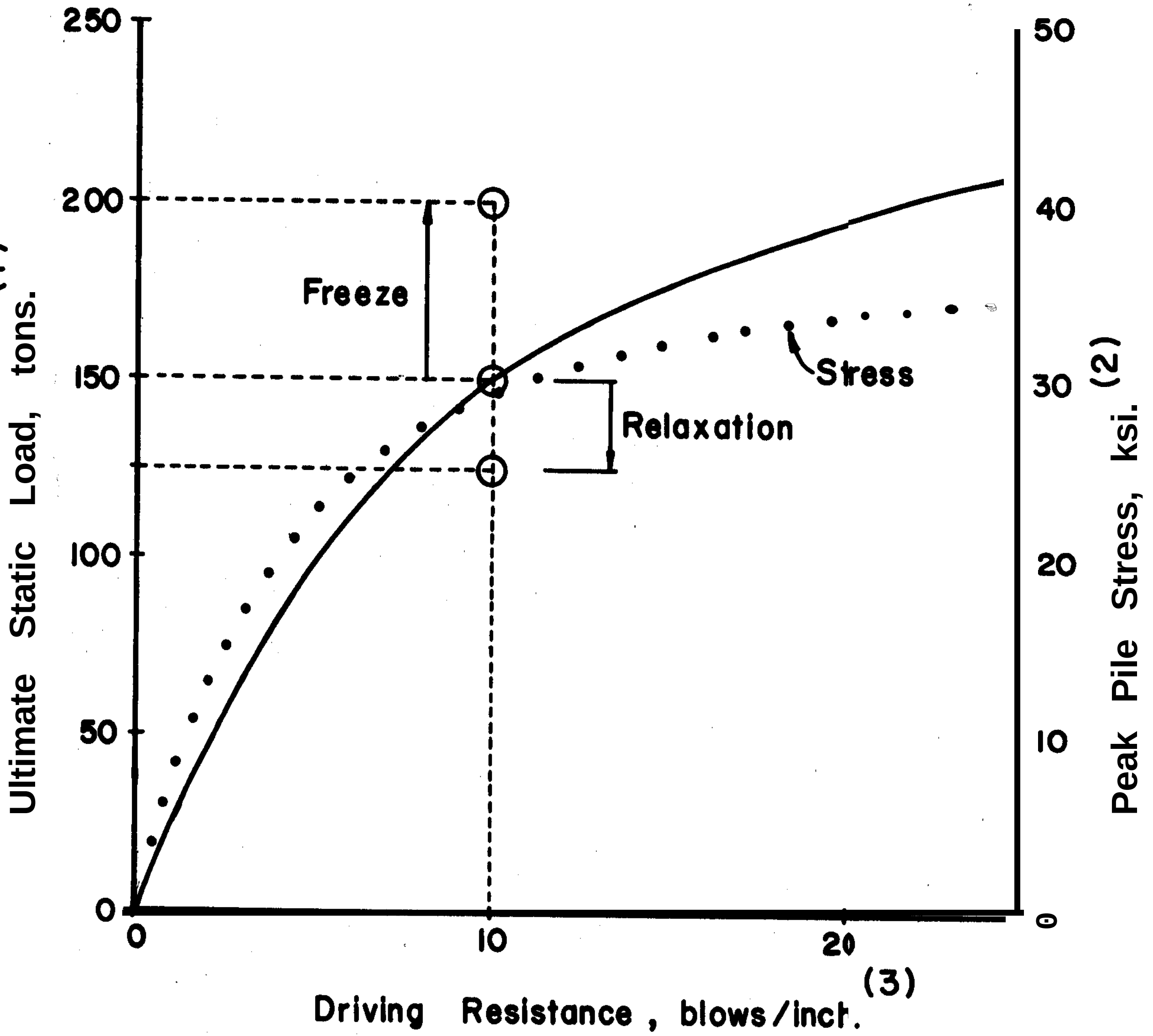


Figure 6- Corrosion of Steel



1. 1 ton equals 8.896 kN.
2. 1 ksi equals 6.895 MPa.
3. 1 inch equals 25.4 mm.

Figure 7. Soil Freeze and Relaxation.

the limits of pile drivability, he has control of the ability to drive for the desired load in the field. If the design calls for an ultimate load beyond that provided by drivability, then the engineer must rely on soil freeze over which he has no control. It is important to distinguish which of the two cases prevails in any given design.

It has been found by Davisson (1972, 1975), that steel piles designed for 12,600 psi (86.9MPa), precast piles designed for 1600 psi (11.0MPa) and timber piles designed for 800 psi (5.5MPa) can always be driven hard enough to develop the resulting pile loads in soil bearing, based on an ultimate load of twice the working or service load. For example, an HP12x53 pile with a cross-sectional area of 15.6 in2 (100.6 cm^2) designed at a working stress of 12,600 psi (86.9MPa) results in a service load of 98 tons (872 kN). Because load testing to twice the service load is normal practice, 196 tons (1744kN) becomes the goal with respect to ultimate load capac ity . Thus, it is possible to find a hammer-cushion combination that will develop twice,98 tons (872kN) in ultimate load capacity (Davisson, 1972) without developing destructive peak driving stresses in the pile.

Thus, there appears to be natural dynamic limits to drivability that are controlled by axial pile stiffness which in turn are controlled by Young's modulus for pile materials; Young 's modulus does not change significantly with increasing material strength. Drivability limitations are most pronounced in steel piles. Further explanation of this behavior is given by Davisson (1975). The allowable stresses derived in this report will be referenced *to* drivability so that it may be determined if soil freeze is likely to be necessary for a satisfactory result in the field.

STRUCTURAL . CONSIDERATIONS

It is well known that even the softest of soils provide sufficient lateral support to piles of normal dimensions that buckling is not the mode of failure in compression. Pile section strength governs. Thus, it is possible to concentrate on short column strength, or strength at a length/radius of gyration ratio equal to zero.

Piles that involve free-standing portions, such as in pile bents, present a special problem. Generally, this is handled in design by considering the below ground portion of the pile under pile strength rules, and the free standing portion of the pile under column rules. This is a special problem that is important, but does not influence the determination of allowable stresses in this report.

In determining allowable stresses it can only be assumed that the resulting pile loads can be developed with respect to the soil on any given project . Thus, soil mechanics considerations are excluded from this study.

Another concept that is discussed by engineers with respect to a group of piles is that of "safety-in-numbers" or the concept of, load sharing. The thought is that one bad pile in a cluster can shed its load to neighboring piles. This may work satisfactorily for interior piles in a large cluster. However, the concept of load sharing in small clusters can be false security. It has been described previously how corner piles and exterior piles in a cluster carry larger than average loads. Should one of these piles prove faulty the tendency is for the pile cap to tilt. In fact, a review of all field tests on groups of piles reveals that the mode of failure is by tilting. Thus, the group of piles may become unusable even though load capacity remains in some of the piles. **One** should not rely upon the load sharing concept in any deliberation on allowable stresses in piles regardless of whether the pile fails by lack of soil support or **by** lack of structural resistance.

HIDDEN DEFECT FACTOR

The discussions of structural member strength found in the literature almost invariably pertain to undamaged members. However, piles are typically subjected to potentially destructive forces during driving such as the forces developed by the hammer and the induced soil/rock resisting forces. Almost all piles are driven somewhat out-of-plumb and with curvatures ; in extreme cases the pile may be bent sharply (dog-leg). Thus, there is need for "a factor that accounts for the environment into which the piles are driven.

Hidden defects are more pernicious in the case of solid piles which cannot be inspected internally after driving. Clearly, some sort of a strength reduction factor should be applied to piles in general, with greater reductions for solid uninspectable piles than for hollow inspectable piles. Three conditions are defined herein for the purpose of allowing the designer an opportunity to exercise judgement on this matter.

Site Condition	Hidden Defect Factor (HDF)
Idea 1	1.00
Normal	0.85
Severe	0.70

Ideal conditions are considered to be soft soils not containing fill or particles larger than gravel size in which the pile penetrates readily under the weight of the hammer or with light driving. The bearing layer should not contain particles larger than gravel size and should not be weak rock into which the pile will penetrate. A resistant rock that causes refusal of the pile may be considered ideal provided pile tip reinforcement is used and driving is controlled so as not to induce damaging stress levels. Soils containing cobbles or larger size material, weak rock into which the pile will penetrate, and uncontrolled fill materials are considered severe conditions.

The hidden defect factors given above can be used as a starting point for rationalizing different values of the factors for different piles. For example, a closed-end steel pipe pile provides a relatively good environment for concrete placement compared to other cast-in-place piles. Therefore, a normal HDF of **0.9** could be assigned instead of the **0.85** indicated above.

SUMMARY

The foregoing discussion of factors influencing pile allowable stresses illustrates the multitude of problems that must be considered. In the final analysis, part of the margin of safety must be determined based on judgement and previous experience because the data necessary for a theoretical analysis of safety is lacking, In the following chapters allowable stresses will be determined for' steel, concrete and timber piles using the information contained herein, 'either directly **or** indirectly; as a reference framework for arriving at the necessary decisions.

CHAPTER FOUR
STEEL PILES

INTRODUCTION

A rational and consistent method for determining allowable pile stresses has been developed, and is used herein. The first items considered are the implications of the methods and rules under which steel piles are purchased. This is followed by a discussion of the relationship of the strength of samples of the material to the strength of the full pile section, and the effects of other material properties. Moment-thrust interaction diagrams are introduced as the method of expressing pile section strength. Hidden pile damage, load factors and other factors are then accounted for in arriving at an allowable stress.

ROLLED STEEL PILES

This category of steel piles consists almost exclusively of H-sections. Considerable research information is available on W-sections used in steel construction; the data is assumed herein to be applicable also to H-piles.

Rules of Purchase - H-piles are usually rolled from ASTM A36 steel (F_y = 36 ksi, 248 MPa) although some use has been made of ASTM A572 Grade 50 steel (F_y = 50 ksi, 345 MPa). The A36 specification for shapes requires a tensile strength between 58,000 and 80,000 psi (400 to 552 MPa), and a minimum yield of 36,000 psi (248 MPa) . Minimum required elongations are also specified along with the chemical requirements. Quality control; however, is covered in ASTM A6 which specifies the tolerance applicable to the items in A36. For example , a tension specimen can be reteste'd if the results do not vary more than 2,000 psi (13.8MPa) on tensile strength, 1,000 psi (6.9 MPa) on yield and 2 percent on elongation. Permissable variations are quoted on chemical requirements and also cross-sectional area and weight where a 2.5 percent variation is allowed; other provisions. cover length, width, thickness, camber, sweep, etc.

Of particular interest are the rules governing yield strength. Two tension specimens are required for each heat. These are longitudinal specimens taken from the web near the flange rather than near the center of the web. both tension specimens must show a minimum yield strength of 36,000 psi (248 MPa) . However, a specimen that tests at least 35,000 psi (241 MPa) can be retested.

Product/Sample *Ratio - Beedle and Tall, (1962) have shown that the yield strength of a stub (short column) column of a W-section is significantly less than the yield point strength of a coupon cut from the section (according to ASTM A6) multiplied by the cross-sectional area of the column. Iwo major reasons exist for the discrepancy, namely, residual stresses in the W-section and the fact that the yield point of a web coupon determined according to ASTM A6 results in a yield stress higher than the average for the entire cross-section.

Residual stresses result from differential cooling during and after the mill rolling operation. In particular, the flange tips cool first and are subjected to compression as the remainder of the section cools and shrinks. It follows, therefore, that the web and portions of the flanges have residual tensions. When a W-section is loaded in compression, the flange tips reach yield prematurely by the amount of the residual compression. After the flange tips yield, the average strain is no longer proportional to average stress and a curved relationship results until yield exists across the full section. This is illustrated in Figure 8, which shows typical results from Beedle and Tall (1962) as assembled by Di smuke 19 78. Note that in Figure 8b the pattern of residual stresses is shown. Also, as the stub column is loaded, the stress-strain relationship, Figure 8a, becomes non-linear as the flange tips yield. A flat-top yield of the stub column occurs at a lower average stress than in a coupon taken from the web. Note that the web coupon's stress-strain relationship is linear until the flat-top yield occurs, in contrast to that for the stub column.

Variations in yield stress of coupons taken from W-sections occur for a variety of reasons, as illustrated in Figure 9. Stress-strain curves A and B illustrate variations that can occur within the tolerances of ASTM A370, the specification controlling physical tests on coupons. Curve A is typical and illustrates an upper yield point up to 10 percent higher than the flat-top yield level. Occasionally a yield point is not observed (curve B); in that case, yield is taken at 0.5 percent strain. By contrast, the same web coupons used for curves A and B produce curve C when tested at near zero strain rate, resulting in a lower yield stress. Thus, a strain rate effect exists in standard testing that results in an overestimate of the static yield stress level.

Variations in yield stress occur with location in the W-section, as illustrated in Figure 9. Curve D for 'the flange exhibits a lower yield stress than curve C for the web. However, the weighted coupon average for various locations in the W-section, curve E, agrees with the stub column test results, curve F. Thus, the coupon upon which a W-section is judged represents a higher than average yield stress and results in an unconservative indication of full section strength.

The available data on the relationship of mill acceptance tests to W-section strengths have been reviewed by several groups, (AASHTO 1977; Bjorhovde, Galambos and Ravindra 1978) with the result that for stub columns, ratios of 0.85 to 0.86 have been proposed as load reduction factors (ϕ - factors). AASHTC uses a ϕ - factor of 0.85, which will be used herein. Considering this discussion in conjunction with that for the rules of purchase, it is clear that a buyer of A36 steel would have to accept a heat if the mill coupons equaled or exceeded 36,000 psi (248 MPa). However, the real section strength would be 15 percent less than indicated by the mill tests. Thus there is a design need for a strength reduction factor ($-factor>; 0.85 is used herein for both bending and compression.

<u>Creep Properties of Steel</u> - Very little to no research data are available on the creep properties of steel at normal temperatures. It is generally considered that creep in mild steel is of sufficiently small magnitude that it can be ignored (Salmon and Johnson, 1980). Creep is, therefore; neglected herein as a consideration.

<u>Size 'Factor</u> - ASTM A6 allows a 2.5 percent deviation in size and weight of rolled W-sections. This factor is too small to merit direct consideration.

<u>Moment - Thrust Interaction Diagrams</u> - The usable strength of a

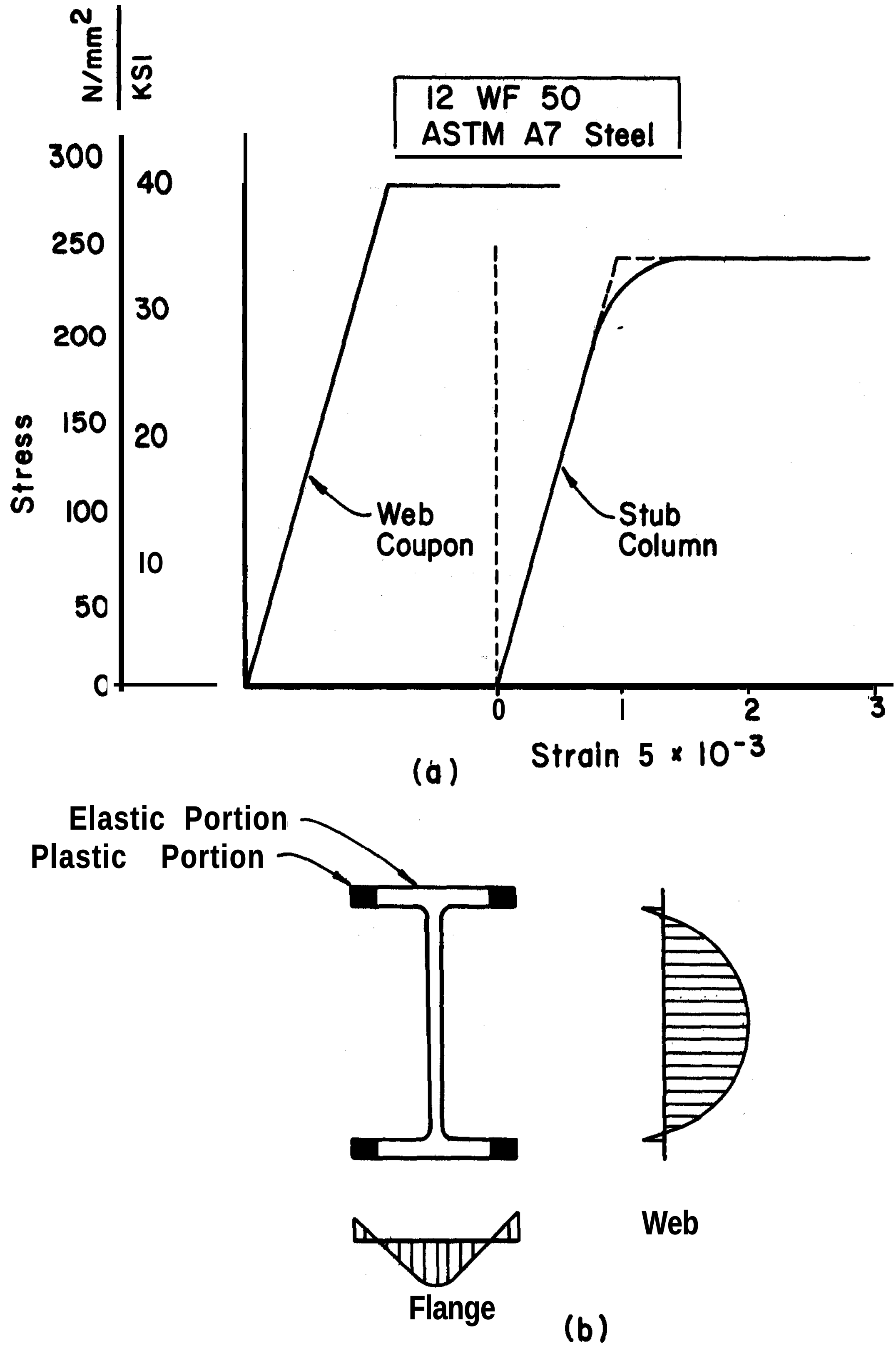
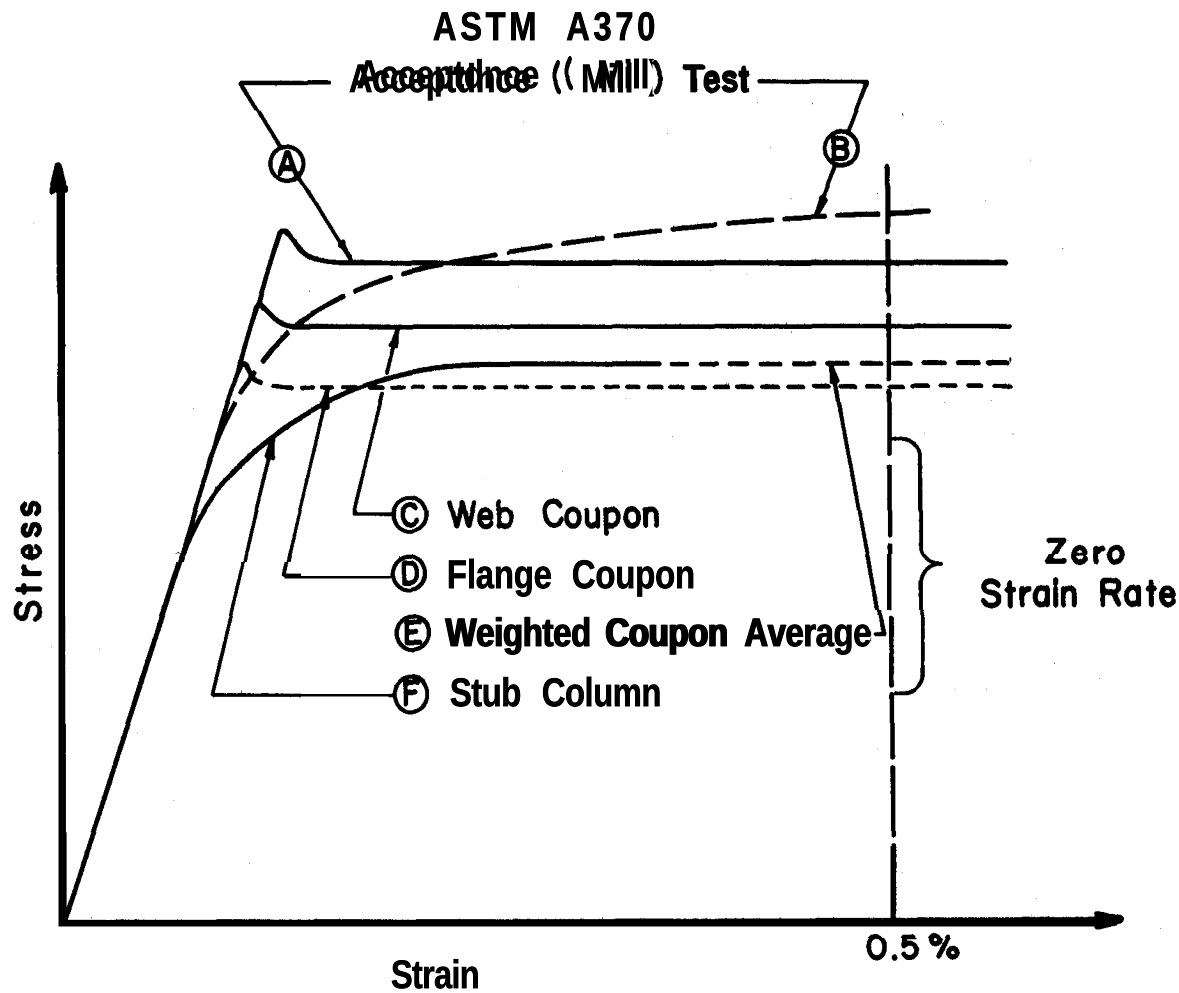


Figure 8. Effects of Residual Stresses.
(After Dismuke 1978)



NOTE:
Ail Tests, Except the Stub Column Test, Were Made on Standard ASTM A370 Tensile Specimens.

Figure 9. Yield Stress Variations..
(After Beedle & Tail, 1962)

beam-column can be best appreciated by use of moment - thrust interaction diagrams as used in concrete design. Use of such diagrams in steel design is imminent because of the recent introduction of plastic design methods. The pile problem is a special case of the beam column because the surrounding soil is almost always of sufficient stiffness to prevent Euler buckling. Thus, embedded portions of a pile may be treated as fully braced for Euler buckling; this leads to a unique moment - thrust relationship. Details of moment - thrust relationships for steel beam-columns are presented by Bjorhovde, Galambos and Ravindra (1978).

Interaction diagrams for a typical H-pile section, both strong and weak axes, are shown on Figures 10a and 10b, respectively. Py is the yield load and is defined as:

$$Py = A\ Fy$$

where A is the cross-sectional area and Fy is the specified yield stress. Me is the moment at zero axial load with the outer fibers at the elastic limit; it is evaluated as:

$$Me = FyS$$

where S is the section modulus. Mp is the fully plastic moment with zero axial load; it is evaluated as:

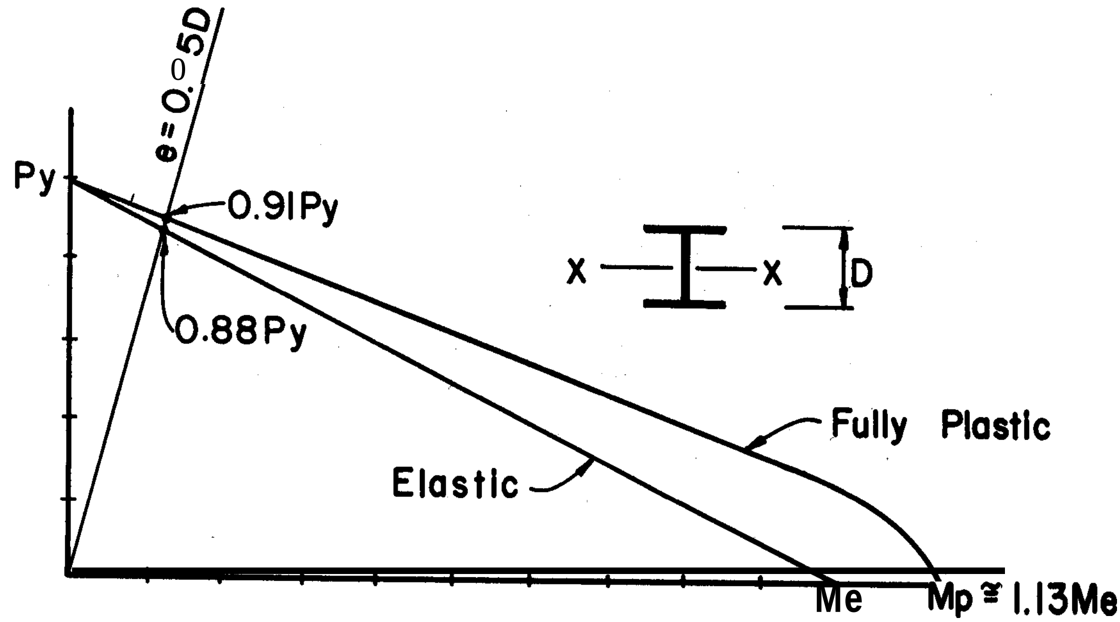
$$Mp = FyZ$$

where Z is the plastic section modulus. For the strong axis, Mp is typically 1.13 Me; for the weak axis, Mp is approximately 1.53 Me. The straight lines drawn from Py to Me represent the limits of fully elastic behavior in the pile. Values of moment and thrust plotting above the elastic line involve at least partial plastic behavior. Values of moment and thrust plotting on the curved lines represent fully plastic behavior of the piles.

Because of the high probability of accidental eccentricities of axial loads good engineering practice has called for a minimum moment for design purposes. ACI followed this practice in the design of concrete columns by using eccentricities of 0.05 times the column width for spiral columns, and 0.10 times the width for tied columns with a minimum of one inch. However, the most recent ACI code (ACI 318-77) simplified the calculation by taking a flat reduction in axial load capacity. The use of a minimum eccentricity is logical for the design of piles. An eccentricity of 0.05 times the depth of the pile section is used here.

It is obvious that an eccentricity from the weak axis controls design. For HP sections, an eccentricity of 0.05 times width typically intercepts the elastic limit at 0.70 Py, and the fully plastic limit at 0.89 Py. For the strong axis, the intercepts are 0.88 Py and 0.91 Py, respectively. For simplicity, axial load capacities above 0.7 Py are considered unusable where the initiation of yield is considered as a limit. Where fully plastic action is the limit; then axial loads above 0.89 Py are considered unusable.

Moment-thrust interaction diagrams for HP sections are sufficiently similar to those for W-sections that simplified approximate non-dimensional forms can be used. Galambos (1968) has presented equations that are plotted on Figure 11. The cross-hatched zone above P/Py of 0.89 represents

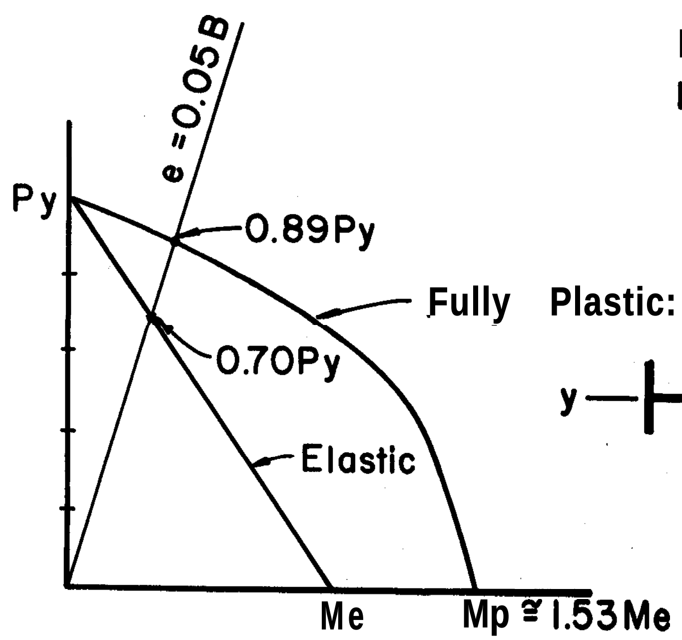


a) Strong Axis

Py = ØFy A

Me = ØFy S

Mp = ØFy Z



b) Weak Axis

Figure 10. Moment - Thrust Interaction Diagrams.

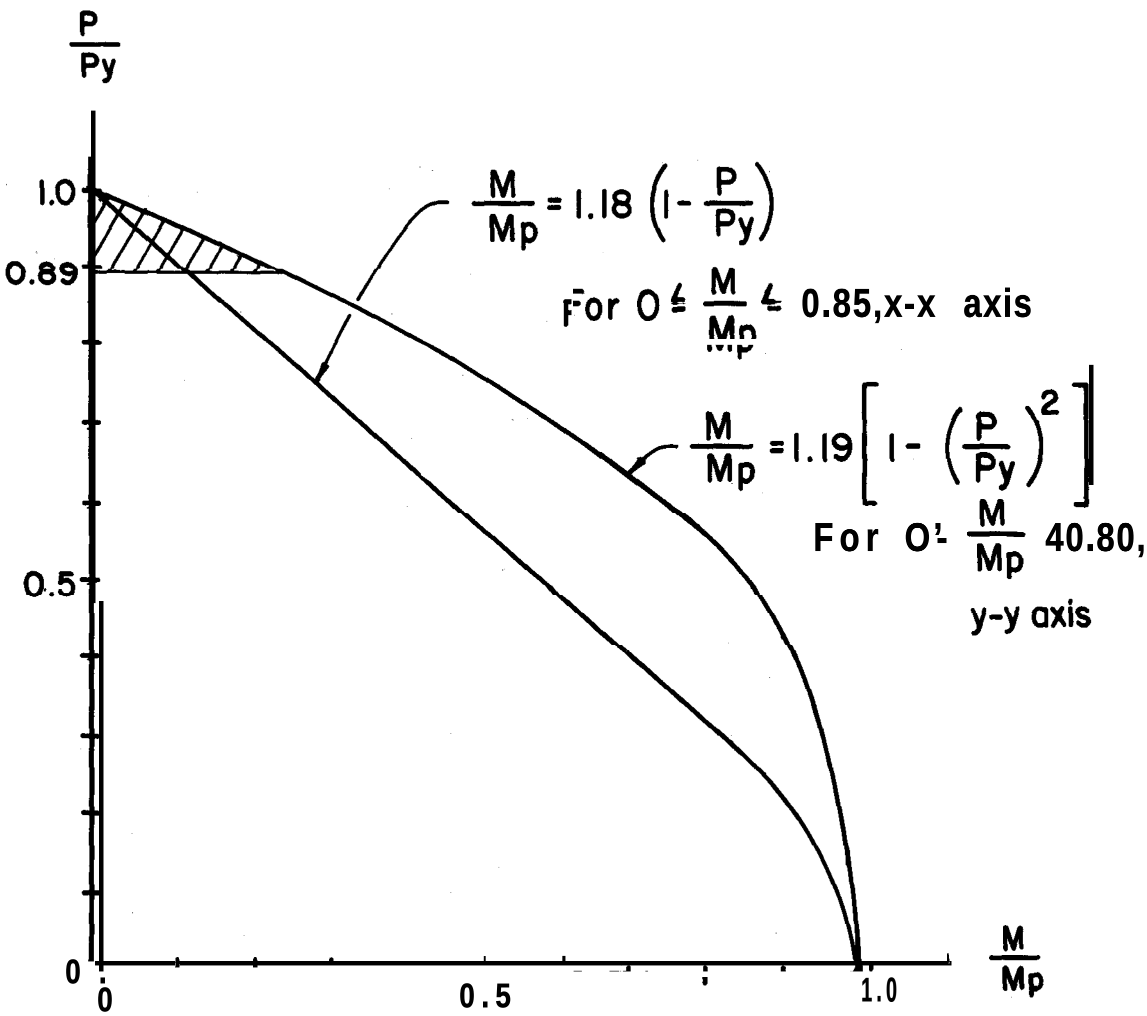


Figure II. Non - Dimensional Moment - 'Thrust Interaction Diagrams for HP Shapes.

the unusable portion of pile strength.

Instability - Normal structural steel beam-column design considerations account for three types of instability in addition to Euler buckling, as follows: (1) local buckling of flanges; (2) web buckling, and (3) lateral-torsional buckling. This is normally set-out in terms of requirements for compact sections. (A compact section is one which is capable of developing its plastic moment capacity before any local buckling occurs.) Soil embedment provides the lateral support necessary to prevent lateral-torsional buckling just as it prevents Euler buckling. Further, the webs of H-sections are relatively thick and web buckling is not a critical mode. The only instability that merits consideration for embedded piles is local (flange) buckling. The soil support near the ground surface cannot be counted on to provide the localized support necessary to prevent flange buckling. In fact, many pile caps exist where soil has settled away from the bottom of the cap causing short unsupported pile sections to exist. Therefore, the normal structural requirements for compact sections should be introduced into considerations of pile allowable stresses.

Flange dimensions are usually limited in compact section criteria. AASHTO requires that b'/t ratios for sections be less than certain limits (AASHTO, 1977, section 1.7 .59) where b' is the width of the unsupported flange and t is the flange thickness. The requirements follow:

Compact	Non-Compact
$F_y=36$, $b'/t \leq 8.4$	$F_y=36$, $b'/t \leq 11.6$
$F_y=50$, $b'/t \leq 7.2$	$F_y=50$, $b'/t \leq 9.8$
$M_u=F_yZ$	$M_u=F_yS$

where Mu is the nominal maximum moment capacity and Z and S are the plastic and elastic section moduli. For a section failing to meet the Non-Compact requirements AASHTO allows reduced capacities by stating that b'/t may be increased $\sqrt{M_u/M}$ by where M is the factored moment load.

The 1978 AISC code is more liberal than AASHTO. Their requirements are:

Compact	Non-Compact
$F_y=36$, $b_f/2t_f \leq 10.8$	$F_y=36$, $b_f/2t_f \leq 15.8$
$F_y=50$, $b_f/2t_f \leq 9.2$	$F_y=50$, $b_f/2t_f \leq 13.4$
$M_u=F_yZ$	$M_u=F_yS$

Where b_f is the flange width and tf is the flange thickness. Appendix C of the code specifies reduction factors for sections not meeting the above requirements.

The net effect of the above requirements is that the plastic interaction diagram can be used for compact H-sections, and the elastic diagram for non-compact H-sections. In some cases H-sections do not meet the non-compact 'requirements and must be subjected to a reduction factor. Table 5 is a summary of the 15 H-sections listed by the steel mills showing the category into which each section falls according to both AASHTO and AISC rules. Under AASHTO rules only the HP 13x100 section meets compact requirements for A36 steel providing the axial load is less than 15 percent of the nominal yield load (FyA). Three sections (HP 14x73, HP 13x60, HP 12x53) fail to meet non-compact section requirements for A36 steel, and'are subject to load reduction coefficients of 0.70 to 0.75 of that for non-compact sections.

TABLE 5. - ROLLRD STRRL H-SECTIONS: COMPACT AND NON-COMPACT

Pile Section	b'/t	$b_f/2t_f$	AASHTO Compact GR 36	AASHTO Compact GR 50	AASHTO Ron-Compact GR 36	AASHTO Ron-Compact CR 50	Reduction To Non-Compact GR 36	Reduction To Non-Compact GR 50	AISC Compact GR 36	AISC Compact GR 50	AISC Non-Compact CR 36	AISC Non-Compact GR 50
HP 14x117	8.8	9.2			X	X			x	X	X	X
HP 14x102	10.0	10.5			X			0.96	x		X	X
HP 14x89	11.5	11.9			X			0.73			X	X
HP 14x73	13.9	14.4					0.70	0.50			X	
HP 13x100	8.1	8.6	X*			X			x	X	X	X
HP 13x87	9.4	9.9			X	x			x		x	X
HP 13x73	11.0	11.5			X			0.79			x	X
HP 13x60	13.5	14.0					0.74	0.53			x	
HP 12x84	8.5	9.0			X	x			X	X	x	X
HP 12x74	9.5	10.0			X	x			x		x	X
HP 12x63	11.3	11.8			X			0.75			x	X
HP 12x53	13.4	13.8					0.75	0.53			X	
HP 10x57	8.6	9.0			X	x			x	X	X	X
HP 10x42	11.5	12.0			X			0.73			X	X
HP 8x36'	8.7	9.2			X	X			x	X	X	X

*Providing the axial load is less than 0.15 FyA

Under AISC rules, 8 of 15 H-sections are compact for A36 steel, whereas 5 of 15 are compact for grade 50 steel. All 15 sections meet non-compact section requirements for A36 steel,, but 3 sections (HP 14x73, HP 13x60, HP 12x53) are subject to reduction for grade 50 steel. Because one of the purposes of this document is to develop pile allowable stresses for highway structures, AASHTO rules will be followed.- Only one H-pile section meets compact section requirements; therefore', it is reasonable to adopt non-compact section rules for specifying the interaction diagram. The straight line interaction diagram will be used herein.

Ridden Defect Factor - A reduction for hidden defects is included herein using the factors introduced in Chapter Three. The hidden defect factors (HDF) used are:

Ideal:	1.00
Norma 1:	0.85
Severe:	0.70

Load Factor - All prior discussion of H-piles was concerned with determining reliable structural strength in'place after driving. To convert this information to an allowable stress for design purposes it is necessary to select a load factor. AASHTO specifies ten cases of loading and fifteen sources of load; therefore, some judgement must be made if a single load factor is to be chosen. Because of the obvious importance of Dead Load and Live Load plus Impact this case of loading, has been selected. AASHTO load' factor design requires 1.3 (1.0D+1.67(L+I)) which means the load factor can vary from 1.3 for dead load only to 2.17 for live load only. The average of these load factors is 1.735.

A load factor of 1.735 is illogical for a practical reason. Prior discussion related to the strength of the pile in place after driving. Normally piles are expected to carry twice the design load as proved by standard pile load test procedures. Therefore, it is logical to design with a load factor exceeding the 2.00 factor that may be applied in a pile load test. A factor of 2.00 appears to be as low as can be justified.

H-Pile Allowable Stresses - The foregoing discussion provides the factors needed to develop allowable stresses for axially loaded H-piles. The allowable stress, fa, may be expressed as:

$$fa = (\phi)(ecc)(HDF)(Fy)/LF$$

Where :

ϕ = ϕ-factor (0.85)

ecc = eccentricity factor (0.70)

HDF = Hidden defect factor (1.0-0.85-0.7)

LF = Load factor (2.00)

Substituting , the expression reduces to:

$$fa = 0.2975(HDF)(Fy)$$

For the various values of the hidden defect factor, the allowable stresses are:

Hidden Defect 'Factor	fa 'as % Fy	Recommended fa
Ideal - 1.0	29.75	0.30 Fy
Normal - 0.85	25.29	0.25 Fy
Severe - 0.70	20.83	0.20 Fy

The third column above gives the recommended values of allowable stress.

Most building codes permit an allowable stress of 0.35 Fy which translates to 12,600 psi (86.9 MPa) for A36, steel. There is a history of success at this stress level, although not without difficulty. The normal recommended value, 0.25 Fy, translates to 9,000 psi (62.1 MPa) for A36 steel, which is the current allowable stress under the AASHTO specification. It can be argued logically that foundations for highway structures should be designed more conservatively than ordinary buildings; the foregoing recommendations are compatible with such logic.

Reductions to the non-compact section stresses are warranted for the three non-conforming sections, as follows:

	Ideal	Normal	Severe
HP 14x73	0.21 Fy	0.18 Fy	0.14 Fy
HP 13x60	0.22 Fy	0.18 Fy	0.15 Fy
HP 12x53	0.22 Fy	0.18 Fy	0.15 Fy

The above stresses are for A36 steel.

STEEL PIPE PILES

Open-end steel pipe piles are seldom used in bridge foundation applications although their use for off-shore structures is quite common. The, majority of the on-shore market for open-end pipe is for underpinning piles which are normally jacked into place; the advantage is the ability to clean-out the interior if necessary to achieve penetration. For embedded piles and for equal allowable stresses it is usually more cost effective to buy steel in the form of H-piles rather than pipe. Only in the case of piles with free-standing portions will it likely be more economical to use pipe rather than H-piles because of the superior column characteristics of pipe sections.

No published information is available on the load-deflection characteristics of pipe stub columns compared with the results of corresponding coupon tests. Hence, no product/sample ratio or $-factor can be developed. It will be assumed herein that a $-factor of 0.85 is applicable to pipe, but this assumption should be reviewed when research data becomes available.

The same procedure used for determining allowable stresses for H-piles is recommended for pipe piles. For open-end pipe the hidden defect factors would also have the same values as for H-piles. The eccentricity factor will be based on 0.05d where d is the pipe outside diameter. As with rolled sections, creep is considered insignificant, and the allowable 5 percent underweight factor will not be considered directly.

Instability - The 1978 AISC code requires d/t ratios not exceeding 3300/Fy (Fy in ksi) where d is the outside diameter and t is the wall thickness. The lowest ratio (73.3) would occur for ASTM A252, Grade 3 (45,000 psi, 310 MPa). AASHTO allows a minimum wall thickness of 0.25 in. (6mm) up to 14 in. (36mm), and 0.375 in. (9mm) at diameters of 14 in. (36mm) or greater. Instability would not be a problem until the diameter exceeds 27 in. (686mm). It is unlikely that a wall thickness as low as 0.375 in. (9mm) would be used at a diameter of 27 in. (685mm); therefore, instability is not a problem and the plastic interaction diagram can be used.

Interaction Diagram - For pipe piles the non-dimensional plastic interaction diagram is a unique curve, as shown on Figure 12. The shape factor for the pipe sections likely to be utilized may be taken as 1.30. Note that at an eccentricity of 0.05d, P/Py has a value of 0.91.

Pipe Pile Allowable Stresses - In a manner similar to that for H-piles the allowable axial stress, fa, may be expressed as:

$$fa = (\phi)(ecc)(HDF)(Fy)/LF$$

where :

ϕ = &factor (0.85)
ecc = eccentricity factor (0.91)
HDF = Hidden defect factor (1.0-0.85-0.7)
LF = load factor (2.00)

Substituting, the expression reduces to:

$$fa = 0.387(HDF)(Fy)$$

For the various values of the hidden defect factor, the allowable stresses are:

Hidden Defect Factor	fa as % Fy	Recommended. fa
Ideal - 1.0	38.68	0.39 Fy
Normal - 0.85	32.87	0.33 Fy
Severe - 0.70	27.07	0.27 Fy

For the grade of steel normally used, namely, ASTM A252 Grade 2, the allowable stresses vary from 9,450 psi (65.2MPa) to 13,650 psi (94.3MPa) which is in the range of allowable stresses currently in use; AASHTO allows 9,000 psi (62MPa).

DRIVING STRESSES

Increase in strength with strain rate can be an important property when considering pile stresses caused by driving. For efficient driving as well as efficient utilization of pile material, it is desirable to stress the pile to the- practical limit during driving. However, this must be tempered by a consideration of low cycle fatigue, because pile driving typically involves on the order of 1000 load repetitions at near-yield stress levels. A review of steel fatigue properties leads to the conclusion that low cycle fatigue is not important in compression. Because steel piles are normally highly stressed in compression relative to tension, fatigue can generally be ignored.

With the general availability of programs for wave equation-analysis of pile driving it is possible to calculate routinely the maximum probable pile stresses induced during driving. Therefore, it is desirable to have some guidelines on what maximum stress is permissible. A fairly significant body of literature exists on the increase of the yield stress with increasing strain rate. For example, Beedle and Tall (1962) have shown that increases of 10 to 15 percent over'the static yield level are noticed in ordinary coupon testing. At higher strain rates the subject is more complicated. Johnson, Wood and Clark (1953), illustrate the concept of delay time before yielding. For a given temperature and strain rate

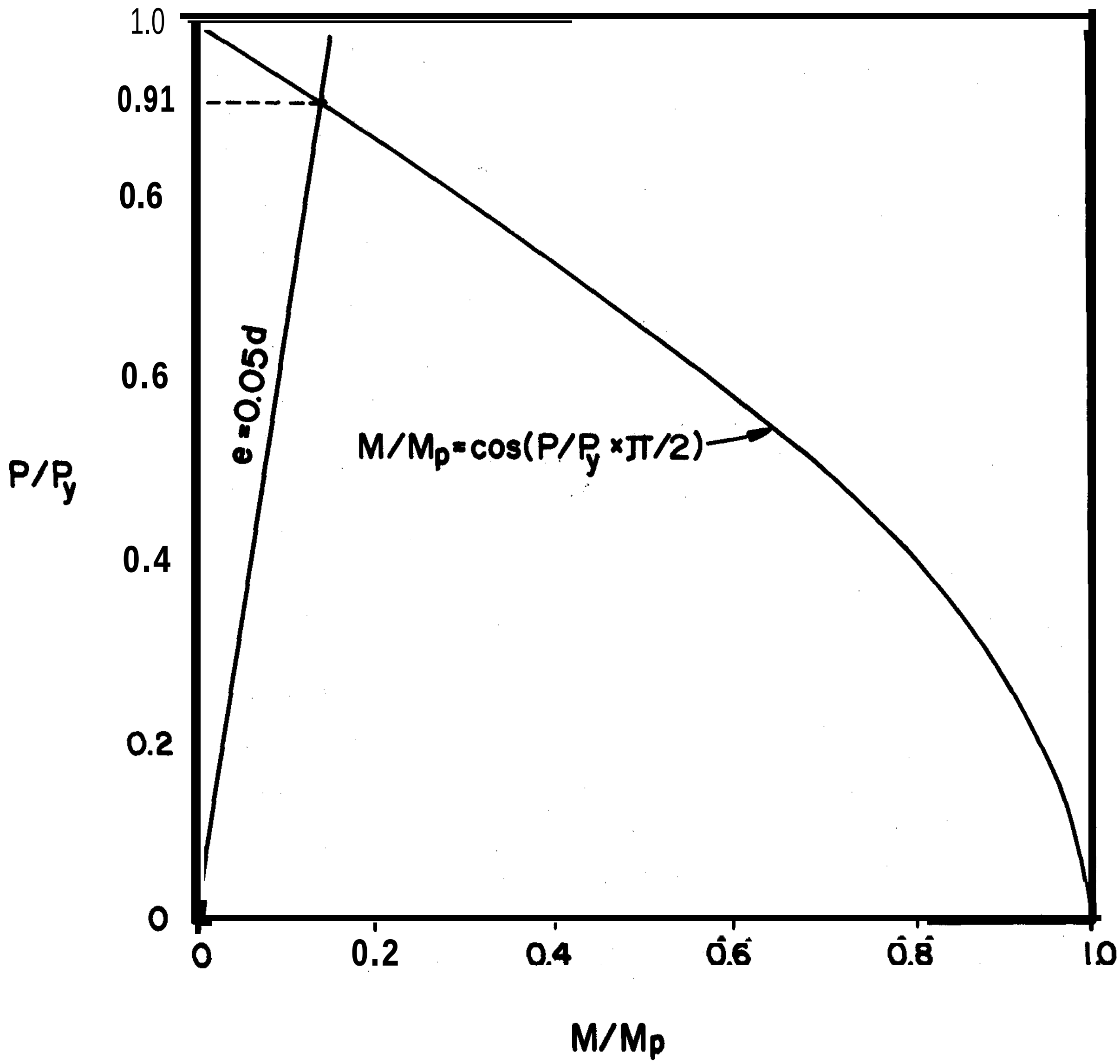


Figure 12. Moment - Thrust Interaction Diagram for Pipe.

there is a delay time for a given stress (above the static yield stress) at which yielding will be initiated. For example, at 1.4Fy a delay of 0.1 millisecond exists before the steel yields.

Selection of an attainable driving stress above the static yield stress may be made by considering the strain-rates associated with pile driving. The temperature and the shapes of force-time pulses encountered in piles during driving are important in considering the applicable delay times. The entire subject is an area worthy of further research. However, until a definitive study is performed it is recommended here in that calculated or measured driving stresses be limited to 1.1 Fy.

ALLOWABLE STRESSES & DRIVABILITY

Davisson (1975) has shown by wave equation analysis that in soil conditions where soil freeze is not experienced, a limit to drivability exists. These limits are discussed in Chapter 3. It should be noted that any structurally derived allowable stress that exceeds the limits of approximately 12,500 psi (86.2 MPa) may not be developed in the field in all cases, regardless of how the pile is driven. This could be the case, for example; with pipe if the 0.39 Fy stress is used, resulting in an allowable stress of 13,650 psi (94.1 MPa) for Grade 2 pipe. Thus, certain precautions should be listed along with structurally derived allowable stresses that exceed 12,500 psi (86.2 MPa). These precautions are:

1. This stress level is likely to be successful only in soils for which the pile skin friction increases significantly after driving.
2. Substantiation in the form of a pile load test, or the results of previous load tests that show the likelihood of success are required before these allowable stresses may be used.

ANALYSIS OF STEEL INDUSTRY RECOMMENDATIONS

The steel industry directs its code lobbying efforts through the American Iron and Steel Institute (AISI). AISI's current position is that the allowable stress of the steel piles should be 0.5Fy. No justification is offered by AISI for their position other than a series of load tests, some of which they sponsored, that were conducted under ideal conditions. Further, the tests were conducted at sites where a large amount of soil freeze is known to exist, which in turn leads to the most optimistic design stresses.

AISI's claim can be appraised by evaluating the B-pile section, for which the following expression is applied herein:

$$fa = \phi\,(ecc)\,(HDF)\,(Fy)/LF$$

AISI claims that ϕ is unity instead of the 0.85 developed in this report, or the 0.86 developed by an American Institute of Steel Construction (AISC) committee working on load and resistance factor design. Further, AISI did not include reduction factors for accidental eccentricity or the items included in the hidden damage factor, HDF.

As pointed out in Chapter Eight, AISI's proposal involves a load factor of 1.19 if both ϕ and accidental eccentricity are considered. Therefore, the AISI proposal appears too bold for buildings, and most certainly is too bold for bridges. The conditions under which allowable stresses that are higher than those recommended herein can be used are detailed in Chapter Eight. However, the upper limit for especially favorable conditions is still below that recommended by AISI for general use.

CHAPTER FIVE
PILES CONTAINING CONCRETE

INTRODUCTION

Ordinary reinforced concrete, prestressed concrete, concrete-filled steel pipe, concrete-filled corrugated mandrel driven shells and auger-grout piles are discussed here. Ordinary reinforced and prestressed concrete piles are essentially identical to short columns used in concrete construction; hence, methods for determining structural strength are well established. The methods of analysis used by the American Concrete Institute (ACI) are well known (Ferguson, 1973) and widely adopted. The AASHTO bridge specification follows essentially ACI methods. Whereas reinforced and prestressed piles are cast before they are driven, the remaining three types of piles are cast after driving.

For convenience, concreted pipe will be referred to herein as pipe, and concreted, mandrel driven, corrugated shells will be called shell piles. The technique for pouring concrete into these piles involves simply casting the concrete into the metal casings. Frequently, the term cast-in-place or cased cast-in-place is applied to this type of pile. Augered piles, on the other hand, are usually concreted by pumping through the hollow stem of the auger as the auger is withdrawn. This type of pile is often referred to as uncased, and the concreting operation is usually denoted as cast-in-situ. A variation of the uncased pile involves driving a metal pipe, concreting, and then pulling the pipe.

The ultimate column compression strength of concrete is universally recognized to be $0.85f'_c$. This product/sample ratio (0.85) is well researched (Richart and Brown, 1934; Richart, et al, 1948). The tension strength of concrete, however, is usually ignored. Steel interacts with concrete in a variety of ways that add to the strength of the member. For reinforcing bars, both tensile and compressive strength is recognized. Because reinforcing bar samples are usually tested full scale to comply with purchase specifications, the product/sample ratio is unity. Prestressing strand is effective in producing compression, its primary purpose, but is also the source of the ultimate tensile strength of the pile. Similar to reinforcing bars, the product/sample ratio is unity. Pipe can contribute significantly to both the ultimate compressive and tensile strength of concrete filled steel pipe piles. As in the case of open-end steel piles a product/sample ratio of 0.85 is used. For corrugated shells the axial strength is considered negligible and is usually ignored. However, shell hoop confining action is very effective in adding to the ultimate axial load, however, and is so recognized in many structural codes. Pipe is also effective in hoop confinement. Structural codes permit pipe to be used either axially or as hoop confinement, but not both.

The discussion of concrete piles that follows describes how the moment-thrust interaction diagrams may be developed for each type of pile. Then the appropriate reduction factors are considered and allowable stresses are developed in much the same manner as for steel piles.

PRECAST CONCRETE PILES

In a report on allowable stresses in concrete piles the Portland Cement

Association (PCA) (1971) derived the ultimate capacity of round and square precast piles with ordinary reinforcement. The expression for nominal ultimate capacity (P_O) of a concentrically loaded short column is:

$$P_O = 0.85f'_cA_c + f_yA_s$$

where A_C and A_s are the cross-sectional areas of concrete and steel, respectively, f'_c is the cylinder strength (28 day), f_y is the yield stress of the reinforcement, and the 0.85 factor is the product/cylinder strength ratio commonly used in concrete design.

The derivation by PCA included an eccentricity of five percent of the diameter or width of the pile. For round piles the ultimate load coefficient for concrete was 0.734 implying an eccentricity factor (ecc) of 0.86 (0.734/0.85). Similarly, for a square pile the coefficient was 0.750 and the eccentricity factor was 0.88. These coefficients and factors vary insignificantly; therefore, single values will be used herein for simplicity. The values selected are 0.734 for the coefficient and 0.87 for eccentricity. These factors are conservative in that the presence of reinforcing steel could cause them to increase perhaps two percent.

Typical moment-thrust interaction diagrams for square and round reinforced concrete piles have been published by Gamble (1979). These are presented as Figures 13 and 14. The diagrams were developed using the assumptions given in the AASHTO (1977) specification.

The spirals and ties normally used in precast piling do not provide the confinement required by ACI and AASHTO for spiral columns. Therefore, the ϕ-factor for tied columns (0.7) is adopted herein. The AASHTC Code (1977) allows a ϕ-factor of unity for factory precast elements and precast piling would usually be factory fabricated. However, the AASHTC requirement was developed for handling of beams where the peak stress the beam sees at any time during its life occurs in handling prior to service, not in service. Therefore, this AASHTO provision will not be applied to piles, and the ϕ-factor of 0.7 is recommended,

Allowable Stresses - In a manner similar to that for steel piles, the allowable load on precast ordinary reinforced concrete piles (Pa) may be expressed as:

$$P_a = (\phi)(ecc)(HDF)(P_o)/LF$$

where

ϕ = +-factor (0.70)

ecc = Eccentricity Factor (0.87)

HDF = Hidden Defect Factor (1.0 - 0.85 - 0.70) from Chapter Three

LF = Load Factor (2.00)

Substituting, the expression reduces to:

$$P_a = 0.3045(P_o)(HDF) = 0.3045(.85f'_cA_c+f_yA_s)HDF$$

For the various values of the hidden defect factor, the allowable loads are:

Hidden Defect Factor	Recommended Allowable Load
Ideal - 1.0	$P_a = 0.26f'_cA_c + 0.30\ f_yA_s$
Normal 0.85	$P_a = 0.22f'_cA_c + 0.26\ f_yA_s$
Severe 0.70	$P_a = 0.18f'_cA_c + 0.21\ f_yA_s$

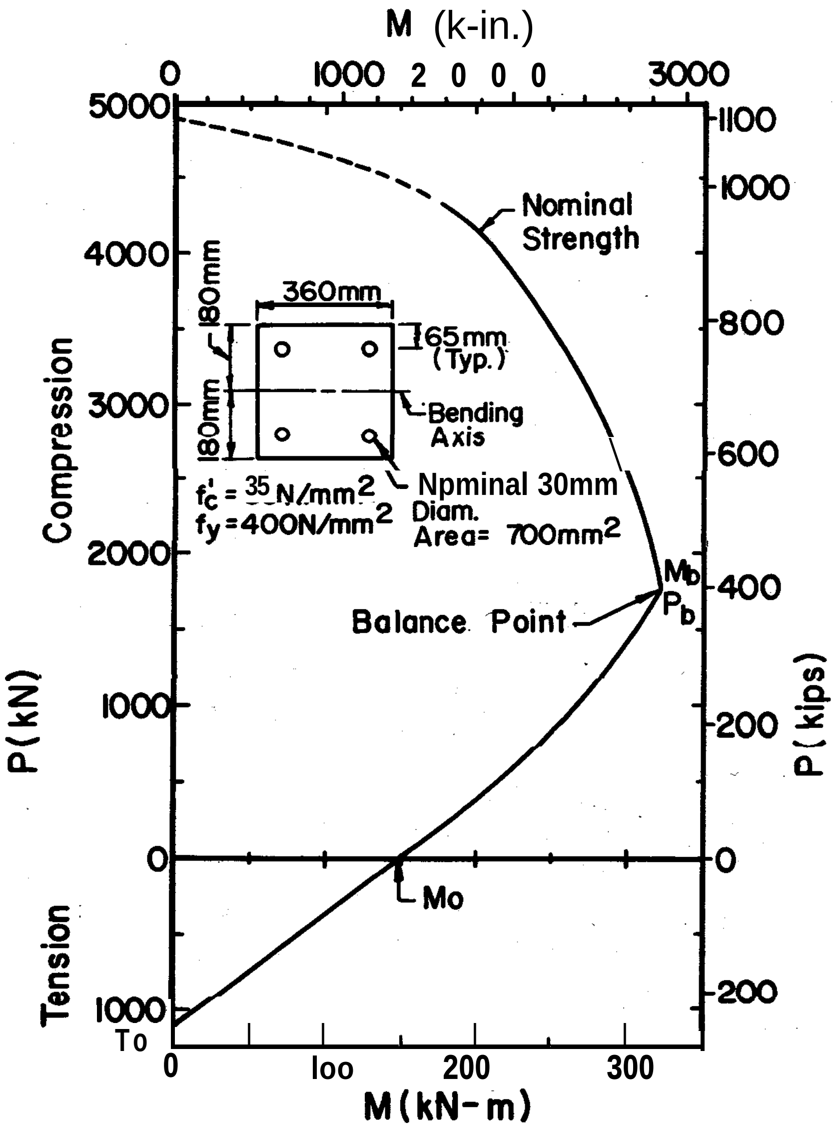


Fig.130Moment - Thrust Interaction Diagmm for **Square** Pile.

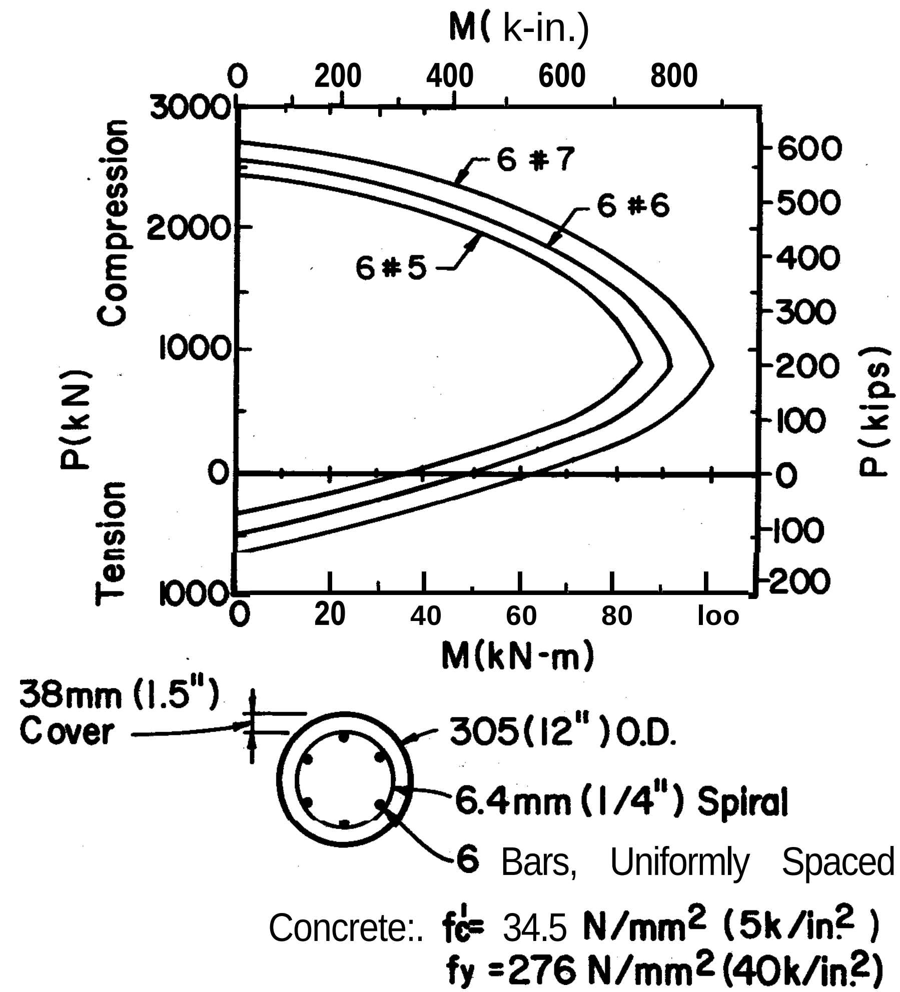


Fig.140Moment -Thrust Interaction Diagrams for Typical Round Piles. (After Gamble, 1979)

Biaxial bending will be dealt with subsequently.

PRESTRESSED CONCRETE PILES

The PCA (1971) report also provides an analysis of prestressed concrete piles. The concentric ultimate axial load (Po) is approximately:

$$Po = (0.85f'c - 0.69fce)\,Ac$$

where fce is the effective prestress in the concrete and Ac is the area of concrete. Using the same factors as for precast piles, the allowable load becomes:

$$Pa = (0.70)\,(0.87)\,(HDF)\,(Po)/2.0$$

which reduces to:

$$Pa = 0.3045(Po)\,(HDF)$$

For the various values of the hidden defect factor, the allowable loads are:

Hidden Defect Factor	Recommended Allowable Load
Ideal - 1.0	$Pa = (0.26f'c - 0.21\ fce)\ Ac$
Normal 0.85	$Pa = (0.22f'c - 0.18\ fce)\ Ac$
Severe 0.70	$Pa = (0.18f'c - 0.15\ fce)\ Ac$

It is noted that the above results are quite similar to the AASHTO-Prestressed Concrete Institute (PCI) Joint Committee (1971) recommendations for allowable load on prestressed piles.

BIAXIAL BENDING OF RECTANGULAR PILES

Section 1.5.33(c) of the AASHTO (1977) code contains a simple method of handling biaxial bending. The nominal ultimate axial load with biaxial bending (Pnxy) may be calculated from:

$$\frac{1}{Pnxy} = \frac{1}{Pnx} + \frac{1}{Pny} - \frac{1}{Po}$$

where Pnx and Pny are the intercepts of the interaction diagrams, at the respective eccentricities, for the x and y axes. This should be used whenever design bending loads on the section exceed the minimum bending based on five percent eccentricity. If the load is primarily bending, then the AASHTO code should be consulted for the appropriate interaction expression.

CONCRETE FILLED STEEL PIPE PILES

The expression commonly used for the ultimate concentric axial load capacity of short concrete filled steel pipe columns is:

$$Po = 0.85f'cAc + FyAs$$

This assumes that the specified steel yield stress, Fy, is available. Consistent with the discussion in the chapter on steel, it is prudent to consider a product/sample ratio for pipe; 0.85 is recommended. Thus, the expression for Po becomes:

$$Po = 0.85\,(f'cAc + FyAs)$$

A study of moment-thrust interaction diagrams for concrete filled pipe is summarized in Figure 15. The diagram shows the combined results of steel and concrete, and also the results for steel alone per the previous discussion of steel pipe. Typically, for concreted pipe the intercept at five percent eccentricity occurs at 0.89 PO.

The subject of ϕ-factors for a pipe column deserves attention. It should be recognized that concrete provides approximately 50 percent of a pipe piles load capacity, and not 80 percent as may be the case in a reinforced concrete column. Thus, a higher degree of reliability is likely. Also, the pipe contains the concrete so that there can be no loss of concrete area, as with the shell of a tied or spiral column. Thus, there is every reason to say ϕ should be at least equal to 0.75, that for a spiral column, and with confirmation by tests this can probably be raised to 0.80. Lacking the requisite data, a ϕ of 0.75 is recommended,

Up till now only solid, internally uninspectable pile sections have been considered. With pipe there is an opportunity to inspect the pile internally to verify it6 structural integrity, Thus, a higher hidden defect factor is appropriate compared to solid pile sections. On the other hand, this is the first pile that has been considered where the concrete has been poured under field conditions. Further, the resulting concrete section is' uninspectable. A mitigating factor is that a severe condition is detectable; thus, this category can be eliminated by either rejection of the pile or individual down-grading by the designer based on field observation. Considering these points, the severe category is eliminated and the remaining hidden defect factors have been selected as 0.9 for normal and 1.0 for ideal conditions.

Using the same expression for working load as for other concrete piles, Pa becomes :

$$Pa = (0.75)(0.89)(HDF)(Po)/2$$

where :

ϕ = 0.75
eccentricity factor = 0.89
load factor = 2.0

This results in the following cases:

Hidden Defect 'Factor	Recommended Allowable Load
Ideal - 1.0	$Pa = 0.28\ (f'cAc + FyAs)$
Normal 0.9	$Pa = 0.25\ (f'cAc + FyAs)$

CONCRETED. SHELL

A non-dimensional moment-thrust interact ion diagram has been developed on the assumption that mandrel-driven corrugated shells do not add strength to a concrete pile section. Thus, Po equals $0.85f'cAc$. The interaction diagram is shown in non-dimensional form as Figure 16. An eccentricity of five percent results in an intercept of 0.89 Po. It is apparent that when an axial compressive force exists the pile possesses some moment capacity. However, at zero axial load the only moment resistance available depends on tension in the concrete, which is ignored herein.

The ϕ -value to be applied to this case clearly cannot exceed that for tied columns (0.7). In fact, there may be no steel whatsoever in this type of pile, and the shell may be corroded away, leaving only plain concrete.

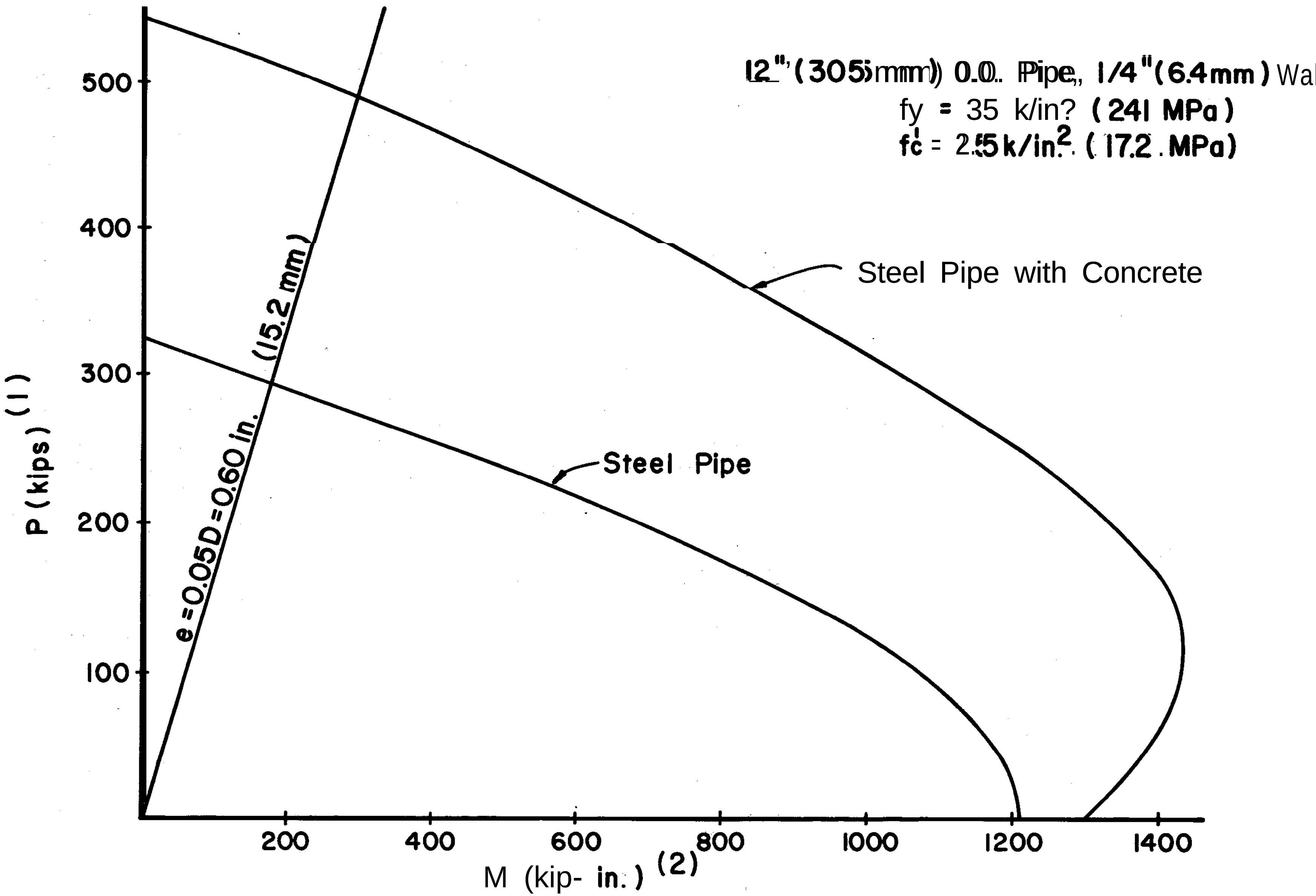


Figure 15. Moment-Thrust Intemction for Concreted Pipe
(1) 1 kip = 4.45 kN
(2) 1 kip-in = 0.113 kN-m

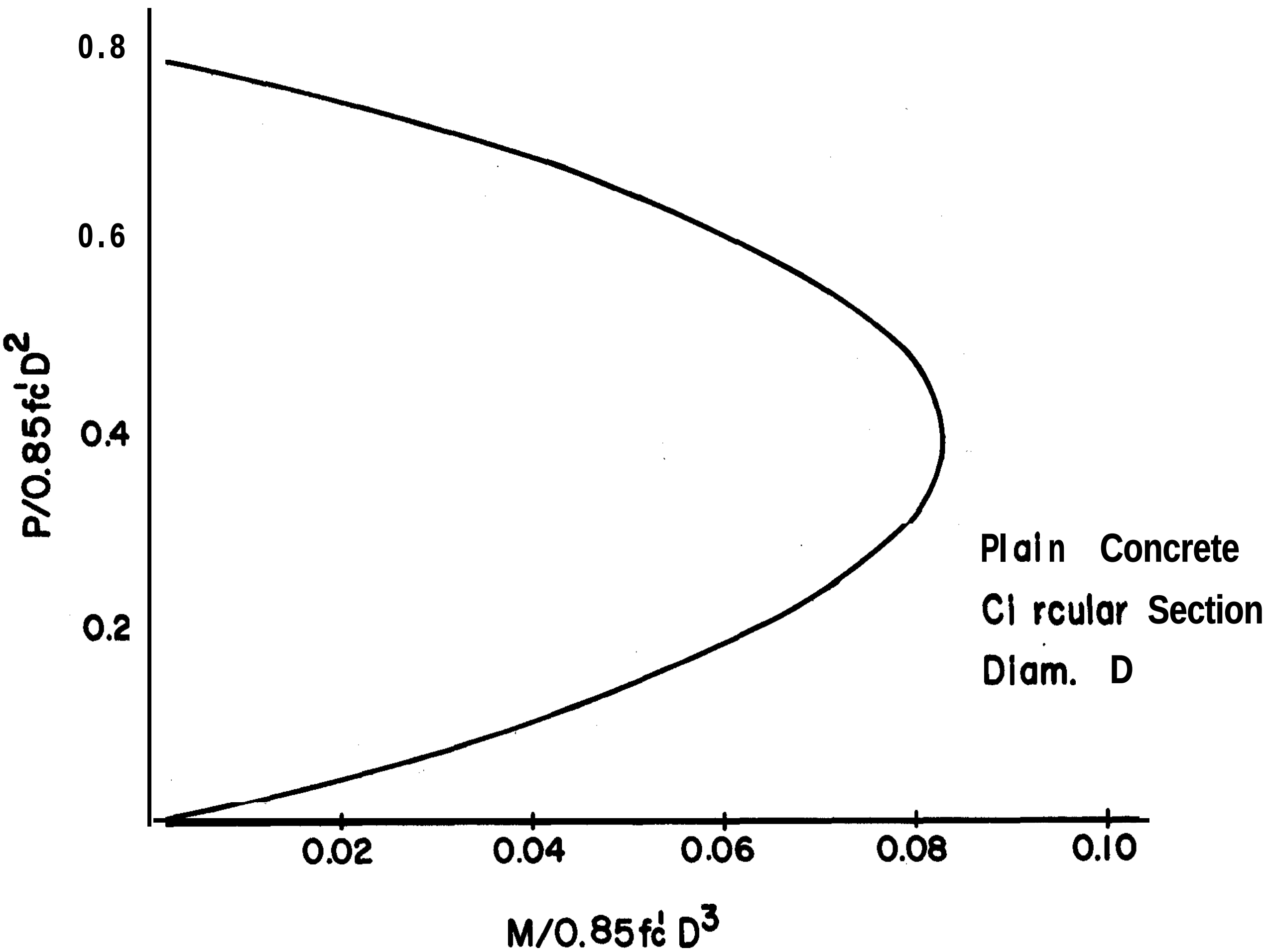


Figure 16. — Moment -Thrust Interaction for Plain Concrete.

Because of the sole dependence on plain concrete it is reasonabe to use a $-factor lower than that for tied columns. A b-factor of 0.65 is recommended and used herein.

Appropriate hidden defect factors also deserve attention. The mandrel-driven shell pile is internally inspectable so that, as with pipe, the severe condition can be detected and obviated. However, the conditions under which the concrete is typically poured into the piles is not as favorable as that for pipe. The corrugations can cause multiple deflection of aggregate as the concrete is poured. Further, one type of mandrel-driven shell involves stepped changes in diameter, providing potential blockages to the flow of the concrete. For these reasons, **0.85** is recommended for the normal hidden defect factor.

Using the same expression for working load as for other concrete piles, Pa becomes:

$$Pa = (0.65)(0.89)(HDF)(Po)/2$$

where : $\phi = 0.65$
HDF = 1.0 (Ideal) and 0.85 (Normal)
Load Factor = 2.00

This results in the following cases:

Hidden Defect Factor	Recommended Allowable Load
Ideal = 1.0	Pa = 0.25 f 'c Ac
Normal 0.85	Pa = 0.21 f 'c Ac

It is noted that considerable satisfactory experience has been accumulated with allowable stresses in the range of 0.20 f'c to 0.25 f'c.

CONCRETED. SHELL WITH CONFINEMENT

One pile driving contractor has promoted the use of shell hoop confinement as a strength increasing factor in mandrel-driven concreted shell piles. The four model building codes (BOCA, NBC, SBC, UBC) have all adopted provisions that increase allowable concrete stress by 20 percent (from **0.33** f 'c to 0.40 f 'c) based on confinement. Considerable literature exists on the subject of concrete filled steel tubes loaded concentrically, but no published literature exists on corrugated concrete filled shells, although there is unpublished data on concentric loading of concreted shells. No data is known to exist on the moment resistance of concreted shells.

The most useful expression for the increase in strength due to confinement is empirical, and was proposed by Richart et al (1928), as follows:

$$fcc = f'c + 4.1p$$

where : fcc = confined concrete strength
p = lateral stress
f'c = strength of the cylinder
4.1 = an empirical coefficient

Subsequent tests have confirmed this expression. Note that the increase in axial stress caused by confinement does not depend on the basic concrete strength. Thus, a given degree of confinement adds more strength percentagewise to low cylinder-strength concrete than it does to high cylinder-strength concrete.

Because Richart's expression dealt with a product/sample ratio of unity, it must be modified to reflect knowledge of the product/sample ratio of columns. Hence, the expression becomes:

$$fcc = 0.85\ f'c + 4.1p$$

The PCA (1971) report on allowable stresses in concrete piles used the above expression in deriving a formula for concentrically-loaded concrete-filled shells with a yield strength, fys. The expression for a pile of diameter, D, and shell thickness, t, is:

$$fcc = 0.85\ f'c\ (1 + 9.65tfys/(Df'c))$$

The PCA report then reduced the 9.65 coefficient to 7.5, a 22 percent decrease. Figure 17 is a nomograph showing the stress recommended by PCA for various gages and diameters of shells. As originally adopted in the codes, the stress is limited to 0.4 f'c with a series of restrictions, as follows:

a. Thickness of shell 14 gage minimum (0.0747 in.-1.89mm).
b. Diameter is not greater than 16 inches (406mm).
c. f'c not over 5000 psi (34.5 MPa).
d. fys/f'c does not exceed 6.

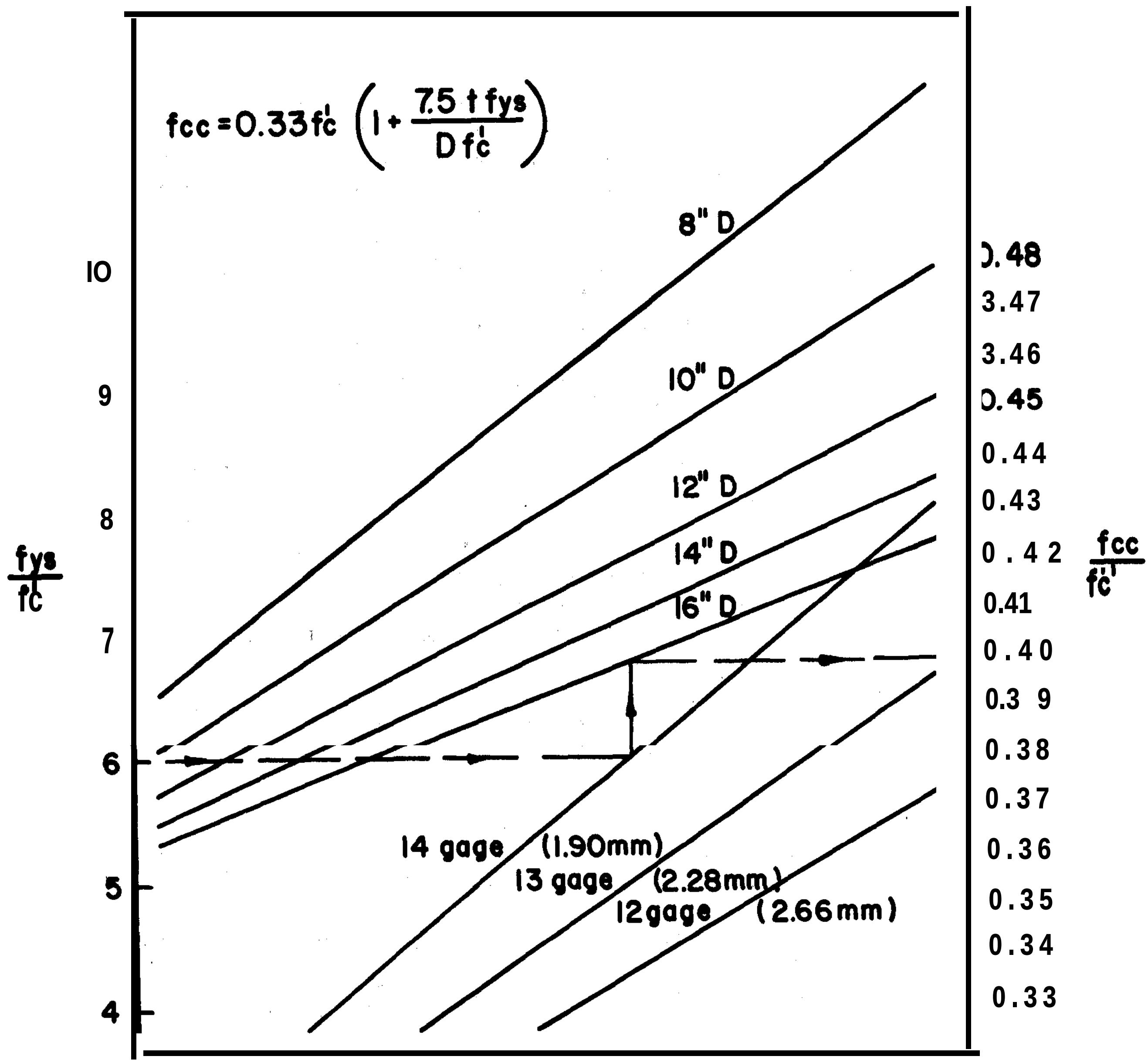
The foregoing restrictions imply that fys does not need to exceed 30;OO0 psi (207 MPa).

There are many factors to be reviewed in evaluating shell confinement. The first item of interest is that ASTM A569 steel (Steel, Carbon, Hot-Rolled Sheet and Strip, Commercial Quality) is used to make shells. This specification has chemical requirements, but no physica 1 requirements ; there is no way to guarantee a minimum yield under this specification. As a consequence, if shell confinement is to be a design consideration, the designer must set up a quality control program involving coupon tests on the shell material actually used, or must specify and acquire pile material under non-standard specifications.

Shell steel acquired according to ASTM A569 has tolerances controlled under ASTM A569. Under these specifications it is possible for shell thickness to be 10 percent less than the nominal value before it is corrugated and welded. Additional deductions should perhaps be taken for the fabrication operations. No data on this subject was available for this study;

Considering that the thickness of shell materials varies from 1/16 in. to 1/8 in. (1.6mm-3mm) it is clear that corrosion cannot be tolerated. This should be a fundamental requirement before shell confinement can be considered in design.

Finally, the test data on confined shells does not address the question of the influence of the shell on bending strength. This is a serious deficiency in documentation and it should be corrected if there is to be a rational basis for structural code provisions on piles involving c onf inement .



1 in = 25.4mm

Figure 17. - PCA Recommended Allowables for Confined Concrete
(After PCA, 1971)

Moment-Thrust **Interaction Diagram** - An attempt is made here to develop an interaction curve. Several assumptions must be made because of the lack of test data. The most likely design use of shell confinement has been selected for illustration. An 8-inch (203mm) shell of 14 ga. (0.0747 in.-1.897mm) steel having a nominal steel yield of 30,000 psi (207 MPa), filled with 5000 psi (34.5 MPa) concrete has been considered. Figure 18 is the interact ion diagram; the solid line represents the contribution of the concrete alone. The dashed lines were drawn from the maximum calculated axial capacity (PO) tangent to the curve for concrete alone; this is an assumption. PO was calculated based on nominal dimensions and yield (coefficient of 9.65 not 7.50 in PCA equation cited above).

The eccentricity factor based on five percent of the diameter is 0.68. This number decreases with increasing shell thickness, and may reach a value below 0.6. It is clear that shell confinement results in a pile section that is sensitive to eccentricity; in many cases more-so than for H-piles.

Because a fundamental design assumption is no shell corrosion, there is no reason to penalize the sect ion with the $-factor for plain concrete (0.65). In reality, the shell does equal or exceed an ACI or AASHTO spiral. Therefore, an increase of 0.05 in the +-factor will be incorporated, resulting in a $-factor of 0.70.

Figure 18 will be used to calculate an allowable load; from this the allowable stress will be back calculated. Using the expressions given previously:

$$P_a = 0.85\, f'_c\, (336/213)\,(0.68)\,(0.70)\,(A_c)\,(HDF)/2$$
$$P_a = 0.319\, f'_c\, (A_c)(HDF)$$

where :

0.68 = eccentricity factor
0.70 = $-factor
336 = PO for 14 gage shell confinement
213 = PO for concrete alone

Using the hidden defect factors developed previously for shells, the results are:

Hidden Defect Factor	Allowable Load
Ideal - 1.0	$P_a = 0.32\, f'_c\, A_c$
Normal 0.85	$P_a = 0.27\, f'_c\, A_c$

It is noted that the results are 28 percent higher than for shells where confinement has been ignored. However, 8/28 of the 28 percent increase was due to raising ϕ from 0.65 to 0.70 because of the design assumption that the shell will not be lost to corrosion. Thus, c onf inement itself may be responsible for a 20 percent increase in the allowable stress.

Considering the gaps in knowledge required to assess confinement properly, it is speculative as to whether confinement with such light gage steel should be permitted. More research is required to provide a sound basis for design procedures involving confinement . In the interim it would be prudent to limit designs involving confinement to perhaps half of the indicated increase in values. Thus, a set of rules could be:

a. f'c not to exceed 5000 psi (34.5 MPa)
b. D not to exceed 16 inches (406 mm)

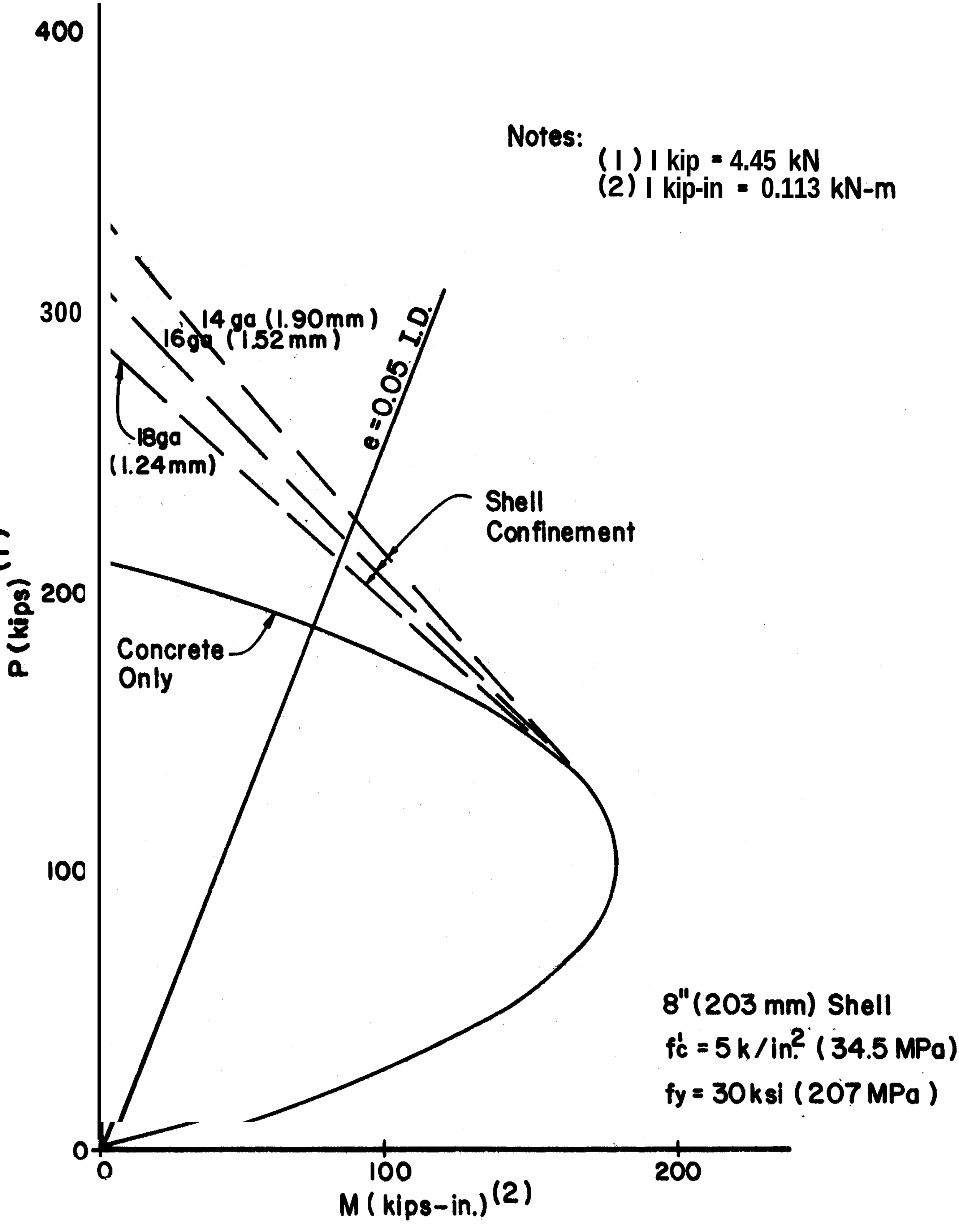


Figure 18. Moment-Thrust Interaction for Confined Concrete.

c. Shell gage 0.0747 in. (1.89 mm) minimum

d. fys to be proven by tests on representative samples

With the above provisions, allowable loads could be:

Hidden Defect Factor	Recommended Allowable Load
Ideal - 1.0	Pa = 0.30 f'c Ac
Normal 0.85	Pa = 0.25 f'c Ac

UNCASED PILES

The uncased or auger-grout pile is similar to the concrete shell pile with respect to the moment-thrust interaction diagram. Thus, Figure 16 applies to the uncased pile, and the eccentricity factor is 0.89.

Considerable thought should be given to selection of an appropriate $-factor. The concreting operation takes place through the stem of a hollow-stem auger as it is withdrawn from the soil, or the concrete is cast in a casing and the casing is pulled. In either case, the pile is not internally inspectable as are cased piles. Many problems have occurred with uncased piles because of poor workmanship, unsatisfactory soil profile for uncased piles, or both. Thus, there is reason to use a lower @-factor than for cased piles. A$-factor of 0.60 is recommended. Thus,

Pa = (0.60) (0.89) (0.85 f'c)(HDF)(Ac)/2
= 0.227 f'c (HDF)(Ac)

where :

0.60 = ϕ-factor
0.89 = eccentricity factor
2.00 = Load factor

Applying the same hidden defect factors used for other internally uninspectable piles, the allowable load becomes:

Hidden Defect Factor	Recommended Allowable Load
Ideal - 1.0	Pa = 0.23f'c Ac
Normal - 0.85	Pa = 0.19f'c Ac
Severe - 0.70	Pa = 0.16f'c Ac

ADDED REINFORCEMENT

For concreted pipe, shell with or without confinement, and the uncased pile it is possible to add longitudinal reinforcing steel to increase load capacity. The presence of such steel can also change the ϕ-factor that was applied, and thus affect allowable stresses. Moreover, the presence of such steel will add an additional term to the expression for ultimate capacity, Po. When a reinforcing bar(s) is added, the term for additional ultimate strength is fyAs reflecting a product/sample ratio of unity. Where a rolled steel section is added, the term for additional ultimate strength is 0.85 FyAs because of the 0.85 product/sample ratio previously described in the chapter on steel piles.

For pipe, additional reinforcement would arguably be sufficient to justify raising ϕ from 0.75 to 0.80. However, at this time ϕ should remain 0.75 because achievement of structural capacity is seldom a problem with pipe piles.

In the case of shell piles the addition of untied reinforcing bars or a rolled structural section does not alter the $-factor. However, a tied reinforcing steel cage inserted into the shell would be sufficient to justify raising ϕ from 0.65 to 0.70. This is reasonable for a full-length cage. A partial-length cage may be justified if it extends through a portion of the soil profile likely to corrode the shell, and the designer is willing to assume that the remainder of the shell remains intact throughout the service life of the structure.

Another point that should enter into judgements on ϕ-factors is that placing reinforcement into small-diameter cast-in-place piles complicates an already difficult concreting problem, and could be detrimental to concrete quality. Thus any increase in sample/product ratios for additional steel may be countered by a decrease in that ratio for the concrete portion of the pile section.

In the case of shell confinement it is arguable that a reinforcing cage would be grounds for increasing ϕ. Inasmuch as ϕ has already been taken as 0.70, no further increase will be considered. Therefore, additional reinforcement is simply an added term within PO.

For uncased piles a tied cage would be grounds for increasing ϕ. However, it is seldom possible to install a full-length cage in such piles, and there is no mitigating factor such as an uncorroded shell to provide confinement. Therefore, no change in ϕ is recommended.,

DRIVING STRES S

The effects of driving stresses can be divided into two categories, which have been studied separately. First is the effect of the stress or strain rate and the second is the effect of a relatively low number of repeated stress cycles at the driving stress level. The pile itself is the principal test specimen in which both effects have been applied at the same time, and that knowledge is largely empirical.

It is well known that high stress or strain rates lead to increased compressive strength in concrete. Stress rates of 21 $kN/mm^2/s$ (3 x 106 $lb/in^2/s$) have been implied from time-force measurements made during pile driving (Davisson 6 McDonald, 1969). On the basis of information from Watstein (1953) and McHenry and Shideler (1956), this stress rate can be expected to lead to a 40 percent increase in compressive strength if the normal, slow-loading strength is 5.5 k/in.2 (38 MPa). It is important to note that the peak stress existed in the pile test measurements for only about 0.001 sec., and time plays an important part in the failure of concrete.

The combined effects of stress rates and fatigue have been studied (Awad and Hilsdorf , 1971). Stress rates comparable to those encountered in pile driving were not reached,. but extrapolations to very high rates were made. It appears reasonable to expect that 73 percent of the single loading strength can be sustained for about 2000 cycles, 80 percent for about 1000 cycles, and 90 percent for no more than 475 cycles. Considering a 40 percent increase due to strain rate, and then 73 percent of that due to fatigue: 1.4 x 0.73 = 1.0 f'c. In simplistic terms, one may conclude that the beneficial effects of having a hi& stress rate'are offset by the low-cycle fatigue effects.

Therefore, a reasonable limit to driving stresses is f 'c as an upper bound; a limit of 0.85 f'c is preferable. It is customary to ignore the tensile strength of concrete. When dealing with prestressed concrete, the prestress must be accounted for in determining net driving stress. For example, a calculated compressive driving stress must be added to the prestress, and the total must be less than the designated limit. Similarly, in tension the calculated tensile stress should be deducted from the prestress to arrive at the net compression; net tensions are undesirable and should be avoided.

ANALYSIS OF CONCRETE INDUSTRY RECOMMENDATIONS

There are no current efforts on the part of a concrete lobbying organization for higher allowable stresses than now exist. However, the PCA (1971) report is the basis for current code values. The expression:

$$fa = \phi(ecc)(HDF)(Po)/LF$$

can be used to evaluate PCA's position.

PCA used both ϕ and an allowance for accidental eccentricity in their derivation. However, no provision was made for hidden damage as was done herein with the HDF. Further, PCA used a load factor of 1.55, whereas 2.00 was used herein. With respect to $-factors, PCA used 0.70, whereas the values used in this report range from 0.60 to 0.75, depending on the type of pile. As discussed in Chapter Eight, the PCA recommendations are very close to the current AASHTO specification. The recommendations made in this report are somewhat more conservative and differentiate between different types of piles. The conditions under which allowable stresses that are higher than those recommended herein can be used are detailed in Chapter Eight. However, the upper limit for especially favorable conditions is likely to approximate current AASHTO (1977) allowable stresses.

CHAPTER SIX
TIMBER PILES.

In the preceding chapters the strength of concrete and steel piles was discussed. Strength of piles manufactured from these man-made materials is at least partially controlled by the designer because he can specify the minimum basic reference strength ($f'c$ or Fy). On the other hand, timber piles are the products of nature and their strength is dictated by nature. In this chapter, a rational and consistent method for determining allowable stresses of timber piles will be developed. This development starts with a general discussion of the large natural variability of clear wood strength, factors influence clear wood strength, and the influence of natural growth imperfections on wood strength. Next, a general background on the design procedures currently used for sawn lumber is presented to illustrate how various factors are considered in wood design. Finally, the general design approach used for sawn lumber is adapted to the special-case of the round timber pile. Where possible, the adaptation will be adjusted by the use of published test data on full size pile sections. Areas are pointed out where further research and testing are required to fill in the gaps in present knowledge of the strength of timber piles.

VARIATIONS IN CLEAR WOOD STRENGTH

Wood that is free of defects or imperfections is referred to as clear wood. The basic strength properties of clear wood are obtained by testing small clear specimens according to ASTM Standard Methods of Testing Small Clear Specimens of Timber (D 143-52). For instance, the tests for compression parallel to grain are made on a 2 by 2 by 8 inch (51 by 51 by 200mm) specimens at a strain rate of 0.003in/in/min. (0.076mm/mm/min.). An idealized stress-strain curve from a compression parallel to grain test on a green small clear specimen is presented in Figure 19. Up to a point called the proportional limit, the stress-strain relation is a straight line, the slope of which is the modulus of elasticity in compression parallel to grain. Beyond the proportional limit, the stress-strain relationship is nonlinear to failure. The unit stress at failure is referred to as the green small clear crushing strength. Typically, the crushing strength is approximately 30 to 40 percent higher than the unit stress at the proportional limit.

The strength properties of clear wood are highly variable. Variability exists not only between trees of different species, but also between trees of one species. In addition, strength properties can vary in different parts of the same tree. If one were to test a large number of green small clear specimens, this variability would be observed. The results of an idealized large testprogram could be presented as a frequency distribution plot as illustrated in Figure 20a. The variation in test results can be closely approximated by a normal frequency distribution, as in Figure 20b, which can be characterized by two variables: s, the average strength and, s_d, the standard deviation. For a normal distribution, 99% of the specimens have a strength greater than $s-2.326s_d$ and 95% will have a strength greater than $s-1.645s_d$. The value $s-1.645s_d$ is referred to as the 5% exclusion strength, and is the basic reference strength used in wood design (Wood Handbook, 1974; Gurfinkel, 1981; Sunley, 1974; Boyd, 1962;

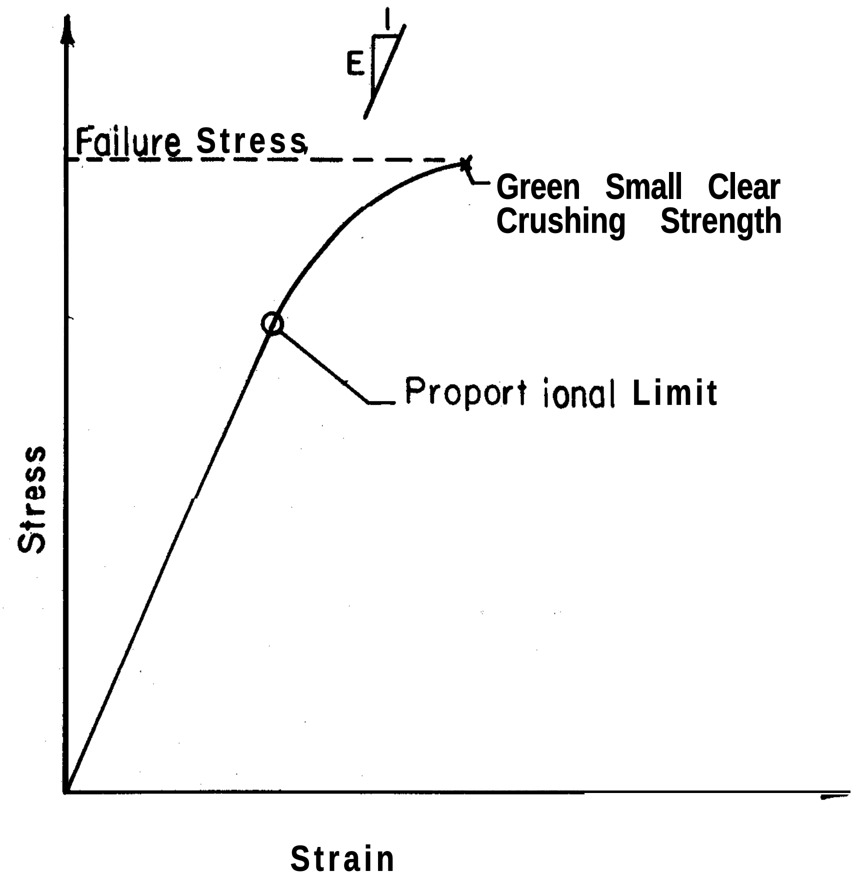
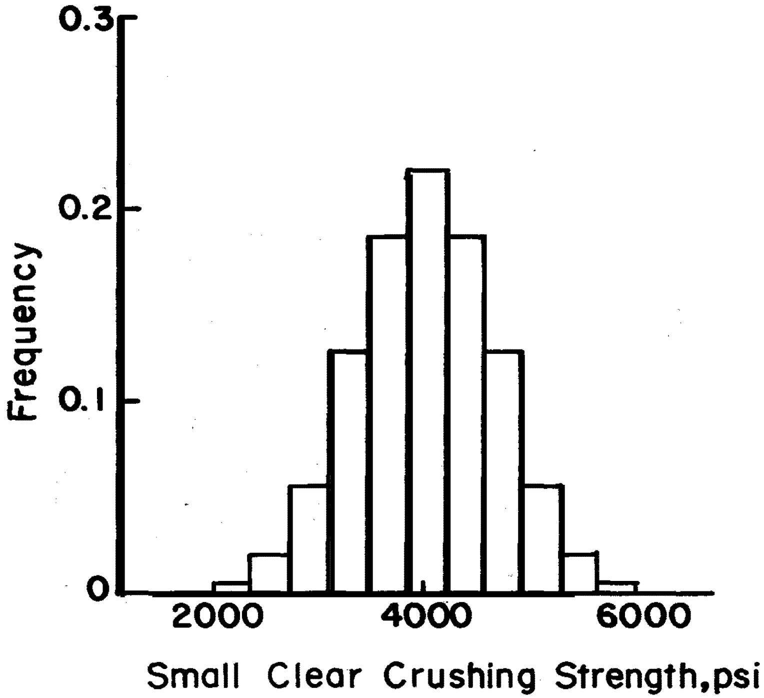
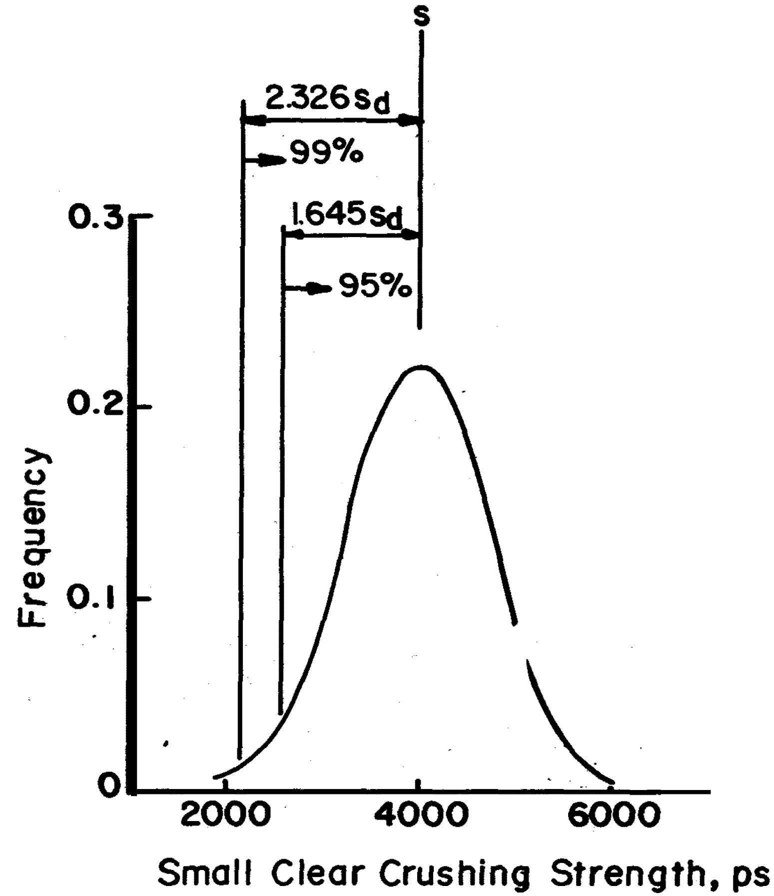


Figure 19: Idealized Stress - Strain Curve for Wood



20a. Idealized Test Results



20b. Equivalent Normal Distribution

Figure 20: Variability of Clear Wood Properties

Timber Construction Manual, 1974).

An ASTM Standard titled method for Establishing Clear Wood Strength Values (D2555-78) summarizes statistical information on the strength of clear wood for various species, and provides procedures for establishing strength values for a group of species that might be marketed under one name. Examples of data presented in ASTM D2555 is shown in Table 6. Information on tensile strength parallel to grain is not presented in Table 6 because that property has not been evaluated extensively. Both ASTM D2555 and the Wood Handbook (1974) recommend that the modulus of rupture be used as a conservative estimate of tensile strength parallel to grain.

ASTM D2555 presents two methods for establishing green clear wood strengths. One method, referred to as Method A, provides for the use of wood density surveys involving extensive sampling of the specific gravity in forest trees in combination with data obtained from standard strength tests made in accordance with ASTM D143. The average strength and standard deviation are obtained from the wood density survey data by using established specific gravity-strength relationships. However, since wood density surveys have been completed for only those species listed in Table 6, they are the only species with which method A can be used.

The second method, referred to as Method B, provides for the establishment of strength values based on standard tests of green small clear specimens for use when data from wood density survey data are not available (species not listed in Table 6). In Method B the standard deviation is estimated by Equation 1.

$$s_d = cs \qquad \text{EQ. 1}$$

where

s_d = standard deviation
s = the average value for the species
c = a coefficient dependent on the strength property being considered

The values of the coefficient, c, given in ASTM D2555 are tabulated below:

Property	c
modulus of rupture	0.16
modulus of elasticity	0.22
crushing strength parallel to grain	0.18
shear strength	0.14
compression perpendicular to grain	0.28
specific gravity	0.10

ASTM D2555 provides tables similar to Table 6 for species which must be evaluated by Method B. The standard deviations for the various properties listed in these tables were determined by Equation 1.

Because of the large variability of properties within a given species, the average properties cannot be used directly as the basic reference properties. In current wood design practice, the basic reference properties for green clear wood is known as the 5 percent exclusion value (ASTM D2555), and represents a value selected so that only 5 percent of a sample of the specimens would have a lower value. The, 5 percent exclusion property, denoted herein by the symbol s', is related to the average and

TABLE 6 - CLEAR WOOD STRENGTH VALUES UNADJUSTED FOR END USE AND MEASURES OF VARIATION FOR COMMERCIAL SPECIES OF WOOD IN THE UNSEASONED CONDITION (METHOD A)

Species or region, or Both	Modulus of Rupture[a] Avg. psi	Modulus of Rupture[a] Variability Index	Modulus of Rupture[a] Standard Deviation, psi	Modulus of Elasticity[b] Avg. 1000 psi	Modulus of Elasticity[b] Variability Index	Modulus of Elasticity[b] Standard Deviation, 1000 psi	Compression Parallel to Grain, crushing strength, max Avg. psi	Compression Parallel to Grain Variability Index	Compression Parallel to Grain Standard Deviation, psi	Shear Strength Avg. psi	Shear Strength Variability Index	Shear Strength Standard Deviation, psi	Compression, perpendicular to Grain;[c] Fiber Stress at Proportional Limit Avg. psi	Compression, perpendicular to Grain Standard Deviation, psi	Specific Gravity Avg. psi	Specific Gravity Variability Index	Specific Gravity Standard Deviation
Douglas fir																	
Coast	7665	1.05	1317	1560	1.05	315	3784	1.05	734	904	1.03	131	382	107	0.45	...	0.057
Interior West	7713	1.03	1322	1513	1.04	324	3872	1.04	799	936	1.02	137	418	117	0.46	...	0.058
Interior North	7438	1.04	1163	1409	1.04	274	3469	1.04	602	947	1.03	126	356	100	0.45	...	0.049
Interior South	6784	1.01	908	1162	1.00	200	3113	1.01	489	953	1.00	153	337	94	0.43	...	0.045
White fir	5834	1.01	949	1161	1.02	249	2902	1.02	528	756	1.01	78	282	79	0.37	...	0.045
California red fir	5809	1.01	885	1170	1.01	267	2758	1.01	459	767	1.00	146	334	94	0.36	...	0.043
Grand fir	5839	1.03	680	1250	1.03	164	2939	1.04	363	739	1.04	97	272	76	0.35	...	0.043
Pacific silver fir	6410	1.07	1296	1420	1.05	255	3142	1.06	591	746	1.05	114	225	63	0.39	...	0.058
Noble fir	6169	1.07	966	1380	1.08	310	3013	1.08	561	802	1.04	136	274	77	0.37	...	0.043
Western hemlock	6637	1.03	1088	1307	1.02	258	3364	1.03	615	864	1.02	105	282	79	0.42	...	0.053
Western larch	7652	1.04	1001	1458	1.02	249	3756	1.04	564	869	1.03	85	399	112	0.48	...	0.048
Black cottonwood	4890	1.00	951	1083	1.00	197	2200	1.00	360	612	1.00	92	165	46	0.31	...	0.034
Southern pine																	
Loblolly	7300	1.08	1199	1402	1.08	321	3511	1.09	612	863	1.05	112	389	109	0.47	1.06	0.057
Longleaf	8538	1.07	1305	1586	1.07	295	4321	1.07	707	1041	1.05	120	479	134	0.54	1.05	0.069
Shortleaf	7435	1.04	1167	1388	1.04	268	3527	1.05	564	905	1.05	125	353	99	0.47	1.05	0.052
Slash	8692	1.09	1127	1532	1.08	295	3823	1.07	547	964	1.05	128	529	148	0.54	1.09	0.062

[a] Modulus of rupture values are applicable to material 2 in. (51 mm) in depth.
[b] Modulus of elasticity values are applicable at a ratio of shear span to depth of 14.
[c] All maximum crushing strength perpendicular to grain values are based on standard test data only.
[d] The regional description of Douglas fir is that given on pp. 54-55 of U.S. Forest Service Research Paper FPL 27, "Western Wood Density Survey Report No. 1."
(From ASTM D2555-78)
1000 psi = 6.9 MPa

standard deviation of that property by Equation 2.

$$s' = s - 1.645 s_d \qquad \text{EQ. 2}$$

where

s' = 5 percent exclusion value for the property
s = average value of the property
s_d = standard deviation for the property

The timber species most frequently used for piles are Douglas fir, red oak, and southern pine. The ASTM D2555 5 percent exclusion values for the crushing strengths parallel to grain, s_c', and the modulus of rupture, s_b', are shown in Tables 7 and 8, respectively, for these three species.

In most cases, timber is marketed as a grouping of species rather than under all the various subspecies headings as listed in Tables 6 through 8. ASTM D2555 provides a procedure for establishing the 5 percent exclusion values for combinations of species marketed under one name. In general the species properties are averaged using a weighting factor based on the standing timber volume of a species in relation to the total standing timber volume of the combination. Tables of timber volume data for some species are provided in ASTM D2555 for this purpose. In addition, ASTM D2555 establishes limitations on the various properties in order to give special consideration to the weaker species included in the combination. Examples of the ASTM D2555 method of determining the 5 percent exclusion value for combinations are presented in Tables 9 and 10 for the combinations of species generally marketed as Douglas fir (Table 9) and Southern Pine (Table 10). Calculations for both the crushing strength parallel to grain and the modulus of rupture are shown.

The first step in the process is to calculate a 5 percent exclusion value for the combined frequency distribution. This is accomplished by adding the areas under the volume weighted frequency distribution of each species, at successively higher property levels, until a value is determined below which 5 percent of the area under the combined frequency distribution will fall. For the examples shown in Table 9, the 5 percent exclusion value for the combined frequency distribution is 2530 psi for crushing strength parallel to grain and 5500 psi for the modulus of rupture.

Once the 5 percent exclusion value for the combined frequency distribution has been found, the composite distribution factor (CDF) for each species in the combination is computed by Equation 3 for groups combined under Method A.

$$CDF = [(s/V.I.) - a]/s_d \qquad \text{EQ. 3}$$

where

CDF = composite distribution factor
s = average value for the property of the species
V.I. = variability index for the property of the species
s_d = standard deviation for the property of the species
a = computed 5 percent exclusion value of the combined frequency distribution of the combination

To prevent weaker species of the combination from reducing the average property of the combination, ASTM D2555 requires that the 5 percent

TABLE 7 - GREEN CLEAR WOOD CRUSHING STRENGTH PARALLEL TO GRAIN

(After ASTM D2555-78)

Species	Average s_c	Standard Deviation s_d	Basic Reference Strength[(b)], s_c'
	Crushing Strength, psi(a)		
DOUGLAS FIR			
Coast	3784	734	2577
Interior West	3872	799	2558
Interior North	3469	602	2479
Interior South	3113	489	2309
OAK, RED			
Black	3470	625	2442
Cherrybark	4620	832	3251
Northern Red	3440	618	2422
Southern Red	3030	545	2133
Laurel	3170	571	2231
Pin	3680	662	2591
Scarlet	4090	736	2879
Water	3740	673	2633
Willow	3000	540	2112
SOUTHERN PINE			
Loblolly	3511	612	2504
Longleaf	4321	707	3158
Shortleaf	3527	564	2599
Slash	3823	547	2923
Pitch	2950	531	2077
Pond	3660	659	2576
Spruce	2835	580	1881
Sand	3440	619	2422
Virginia	3420	616	2407

(a.) 1000 psi = 6.9 MPa
(b.) Based on Equation 2

TABLE 8 - GREEN CLEAR WOOD MODULUS OF RUPTURE

(AFTER ASTM D2555-78)

Species	Average s_b	Standard Deviation s_d	Basic Reference Strength(b), s_b'
	Modulus of Rupture, psi(a)		
DOUGLAS FIR			
Coast	7665	1317	5499
Interior West	7713	1322	5538
Interior North	7438	1163	5525
Interior South	6784	908	5290
OAK, RED			
Black	8220	1315	6057
Cherrybark	10850	1736	7994
Northern Red	8300	1328	6115
Southern Red	6920	1107	5099
Laurel	7940	1270	5851
Pin	8330	1333	6137
Scarlet	10420	1667	7678
Water	8910	1426	6564
Willow	7400	1184	5452
SOUTHERN PINE			
Loblolly	7300	1199	5328
Longleaf	8538	1305	6391
Shortleaf	7435	1167	5515
Slash	8692	1127	6838
Pitch	6830	1093	5032
Pond	7450	1192	5489
Spruce	6004	1102	4191
Sand	7500	1200	5526
Virginia	7330	1173	5400

(a.) 1000 psi = 6.9 MPa
(b.) Based on Equation 2

TABLE 9 - 5 PERCENT EXCLUSION VALUES FOR DOUGLAS FIR (AFTER ASTM D2555-78)

9a. Compression Parallel to Grain

Species	Average Crushing Strength psi	Variability Index	Standard Deviation psi	5% Exclusion Value for Species psi	Volume, Million Cubic ft.	Percent of Total Volume	Composite Dispersion Factor (CDF)
Coast	3784	1.05	734	2577	58878	53.59	1.463
Interior West	3872	1.04	799	2558	26602	24.21	1.493
Interior North	3469	1.04	602	2479	20408	18.57	1.338
Interior South	3113	1.01	489	2308	3987	3.63	1.129

5 Percent Exclusion value for combination of Species by adding areas of volume weighted frequency diagrams for each species = 2530 psi. However, since the CDF for Interior South is less than 1.18, a 5 percent exclusion value = 3113/1.01 - 1.18 (489) = 2505 psi is the maximum assignable to the combination.

9b. Modulus of Rupture

Species	Average Modulus of Rupture psi	Variability Index	Standard Deviation psi	5% Exclusion Value for Species psi	Volume, Million Cubic ft.	Percent of Total Volume	Composite Dispersion Factor (CDF)
Coast	7665	1.05	1317	5499	58878	53.59	1.367
Interior West	7713	1.03	1322	5538	26602	24.21	1.504
Interior North	7438	1.04	1163	5525	20408	18.57	1.420
Interior South	6784	1.01	908	5290	3987	3.63	1.340

5 Percent Exclusion value for combination of species by adding areas of volume weighted frequency diagrams for each species = 5500 psi. Since the CDF for all species are greater than 1.18, the 5 percent exclusion value calculated, 5500psi, is assigned to the combination.

Note: 1000 psi = 6.9 Mpa.

TABLE 10 5 PERCENT EXCLUSION VALUES FOR SOUTHERN PINE (AFTER ASTM D2555-78)

10a. Compression Parallel to Grain

Species	Average Crushing Strength psi	Variability Index	Standard Deviation psi	5% Exclusion Value for Species psi	Volume, Million Cubic ft.	Percent of Total Volume	Composite Dispersion Factor (CDF)
Loblolly	3511	1.09	612	2504	27610	50.67	1.030
Longleaf	4321	1.07	707	3158	5534	10.16	2.047
Shortleaf	3527	1.05	564	2599	16328	29.97	1.362
Slash	3823	1.07	547	2923	5017	9.21	1.795

5 Percent Exclusion value for combination of species by adding areas of volume weighted frequency diagram for each species = 2591 psi. Since CDF for Loblolly is less than 1.18, a 5 percent exclusion value of 3511/1.09 - 1.18 (612) = 2499 psi is the maximum assignable to the combination.

10b. Modulus of Rupture

Species	Average Modulus of Rupture psi	Variability Index	Standard Deviation psi	5% Exclusion Value for Species psi	Volume, Million Cubic ft.	Percent of Total Volume	Composite Dispersion Factor (CDF)
Loblolly	7300	1.08	1199	5328	27610	50.67	1.046
Longleaf	8538	1.07	1305	6391	5534.	10.16	1.896
Interior North	7435	1.04	1167	5515	16328	29.97	1.409
Interior South	8692	1.09	1127	6838	5017	9.21	2.191

5 Percent Exclusion value for combination of species by adding areas of volume weighted frequency diagram for each species = 5505 psi. Since the CDF for loblolly is less than 1.18, a 5 percent Exclusion value of 7300/1.08 = 1.18 (1199) = 5344 psi is the maximum assignable to the combination.

Note: 1000 psi ≈ 6.9 MPa

exclusion value assigned to a combination of species usiug Method A be limited to a value resulting in a composite distribution factor of not less than 1.18. Hence, if the CDF for any species in the combination computed by EQ. 3 results in a value less than 1.18, the maximum 5 percent exclusion value (s') assignable to the combination is given by Equation 4.

$$s' = (s/V.I.) - 1.18s_d \qquad \text{EQ. 4}$$

If more than one species in the marketed combination has a CDF less than 1.18, the 5 percent exclusion value assigned to the combination is the lowest value computed by Equation 4 for the species with CDF-values less than 1.18. On the other hand, if all species within a group have computed CDF-values greater than 1.18, the 5 percent exclusion value assigned to the group is the 5 percent exclusion value of the combined frequency distribution, s' = a.

A review of Table 9 indicates that the assignable 5 percent exclusion value for crushing strength parallel to grain is 2505 psi for the Douglas fir combination and is controlled by the strength properties of the interior south subspecies, whereas the assignable 5 percent exclusion value for the modulus of rupture for the Douglas fir combination is 5500 psi which was calculated from the combined frequency distribution. For the southern pine combination (Table 10) the assignable 5 percent exclusion values are 2499 psi and 5344 psi for crushing strength parallel to grain and modulus of rupture, respectively; in both cases, the assignable 5 percent exclusion values for the combination is controlled by the loblolly subspecies.

FACTORS INFLUENCING CLEAR WOOD STRENGTH

The basic design reference strength, s', referred to as the 5 percent exclusion value, is analagous to design material strengths f'_c and F_y used in concrete and steel design, respectively. With steel and concrete the derivation of design stresses proceeds directly from the basic reference strengths by applying strength reduction factors, load factors, sample/product ratios, and eccentricity factors. With wood design it is necessary to include an intermediate step which adjusts small gree'n clear strengths to in-use clear wood strengths. This includes consideration of in-use moisture content, temperature, and load duration,

Moisture&Content - Most wood properties in Table 6 are affected by changes in moisture content below a moisture content slightly less than the fiber saturation point, which is approximately 30 percent. The moisture contents at which the properties start to change, referred to herein by the symbol M_p, is approximately 24 percent for Douglas fir, 21 percent for southern pine, and 25 percent for oak (Wood Handbook, 1974). For moisture contents greater than M_p, the properties are equal to the green wood properties . ASTM D2555 lists ratios of clear wood properties at 12 percent moisture content to the properties for green clear wood. These ratios are listed in Table 11 for the Douglas fir, southern pine and red oak species. For moisture contents less than M_p the strength can be estimated by the following formula (Wood Handbook 1974):

TABLE 11 - RATIO OF CLEAR WOOD PROPERTY AT 12% MOISTURE TO GREEN CLEAR WOOD PROPERTY (AFTER ASTM D2555-78)

Species or Region, or Both (Official Common Tree Names)	Modulus of Rupture	Modulus of Elasticity	Compression Parallel to Grain Crushing Strength, max.	Shear Strength	Compression Perpendicular to Grain Fiber Stress at Proportional Limit
Douglas Fir					
Coast	1.62	1.25	1.91	1.25	2.08
Interior North	1.76	1.27	1.99	1.48	2.16
Interior South	1.75	1.28	2.00	1.59	2.20
Interior West	1.64	1.21	1.92	1.38	1.82
Pine, southern yellow					
Loblolly	1.75	1.28	2.03	1.61	2.04
Longleaf	1.70	1.25	1.96.	1.45	2.01
Pitch	1.59	1.19	2.01	1.58	2.23
Pond	1.56	1.37	2.06	1.48	2.06
Sand	1.54	1.38	2.01	.96	1.86
Shortleaf	1.76	1.26	2.06	1.54	2.31
Slash	1.87	1.29	2.13	1.74	1.93
Spruce	1.73	1.23	1.99	1.66	2.63
Virginia	1.77	1.25	1.96	1.52	2.32
Oak, red					
Black	1.69	1.39	1.88	1.56	1.32
Cherrybark	1.67	1.27	1.89	1.51	1.63
Laurel	1.59	1.21	2.20	1.55	1.85
Northern red	1.72	1.35	1.97	1.46	1.65
Pin	1.69	1.31	1.85	1.61	1.42
Scarlet	1.67	1.30	2.04	1.34	1.34
Southern red	1.58	1.31	2.01	1.49	1.60
Water	1.72	1.30	1.81	1.63	1.65
Willow	1.96	1.48	2.35	1.40	1.85

$$P/P_g = (P_{12}/P_g)^{[1-(M-12)/(M_p-12)]} \qquad \text{EQ. 5}$$

where :

- P = property value at moisture content M
- P_g = property value in green condition
- P_{12} = properly value at 12% moisture
- M_p = moisture content at which strength starts to change
- M = moisture content at which the property P is to be determined

The above increase in strength at moisture contents below M_p assumes small clear specimens. The indicated increase in strength and modulus of elasticity may not be realized in larger structural members because of the occurrence of drying defects and shrinkage that accompany drying.

Temperature - The data presented in ASTM D2555 are derived from tests in accordance with ASTM D143, and are applicable to temperatures of 68 ± 6°F (17° to 23°C). , In general; the strength-properties of wood tend to increase when cooled below this temperature and decrease when heated. If wood is quickly cooled or heated and then tested at that temperature an "immediate effect" would be observed, resulting in an increase in strength when cooled or a decrease in strength when heated. This "immediate effect" is reversible, except at extreme ends of the temperature scale such that when the specimens are returned to normal temperature no permanent strength change results. The "immediate effect" of temperature on strength is illustrated in Figure 21, where the strength as a percent of the strength at normal temperature (68°F, 20°C) is shown for wood at zero and 12 percent moisture content (Wood Handbook, 1974). The various properties are changed different amounts dependent on the moisture content. The immediate effect is generally reversible for temperatures up to approximately 150° to 200°F (65° to 930C) (Panshin and DeZeeuw, 1980).

When wood is heated to temperatures above 150° to 2000F (65° to 93oC) and then tested at normal temperature (68°F, 20°C) "permanent" property reductions are observed. The amount of irreversible property loss is dependent on the temperature, exposure period, heating medium, moisture content, size of the wood piece, and the property being considered (Hunt and Garratt, 1953; Wood Handbook, 1974; and Panshin and DeZeeuw, 1980). The "permanent" effect at elevated temperatures is cumulative when subjected to several cycles of heating. Examples of the "permanent" property losses observed when wood is heated, cooled to normal temperature, and conditioned to a moisture content of 12 percent are shown in Figures 22 and 23.

The examples of strength loss shown in Figures 21 through 23 are for wood in a dry condition. The effects of temperature are dependent on the moisture content, with the observed property loss increasing as the moisture content increases. Hence, the property loss for green wood subjected to elevated temperatures could be considerably greater than indicated in Figures 21 through 23. Also, if the wood was heated to a temperature above 150° to 2000F (65° to 93° C) and then tested at that elevated temperature, the observed strength loss would be composed of an "immediate" and "permanent" component (Wood Handbook, 1974).

Duration of Loading - The unit stress at which a wood specimen fails is dependent on the time duration the specimen is subjected to the stress. In the standard test procedure (ASTM D143), the loading rate is such that failure is reached in approximately 2 to 5 minutes. If the load was

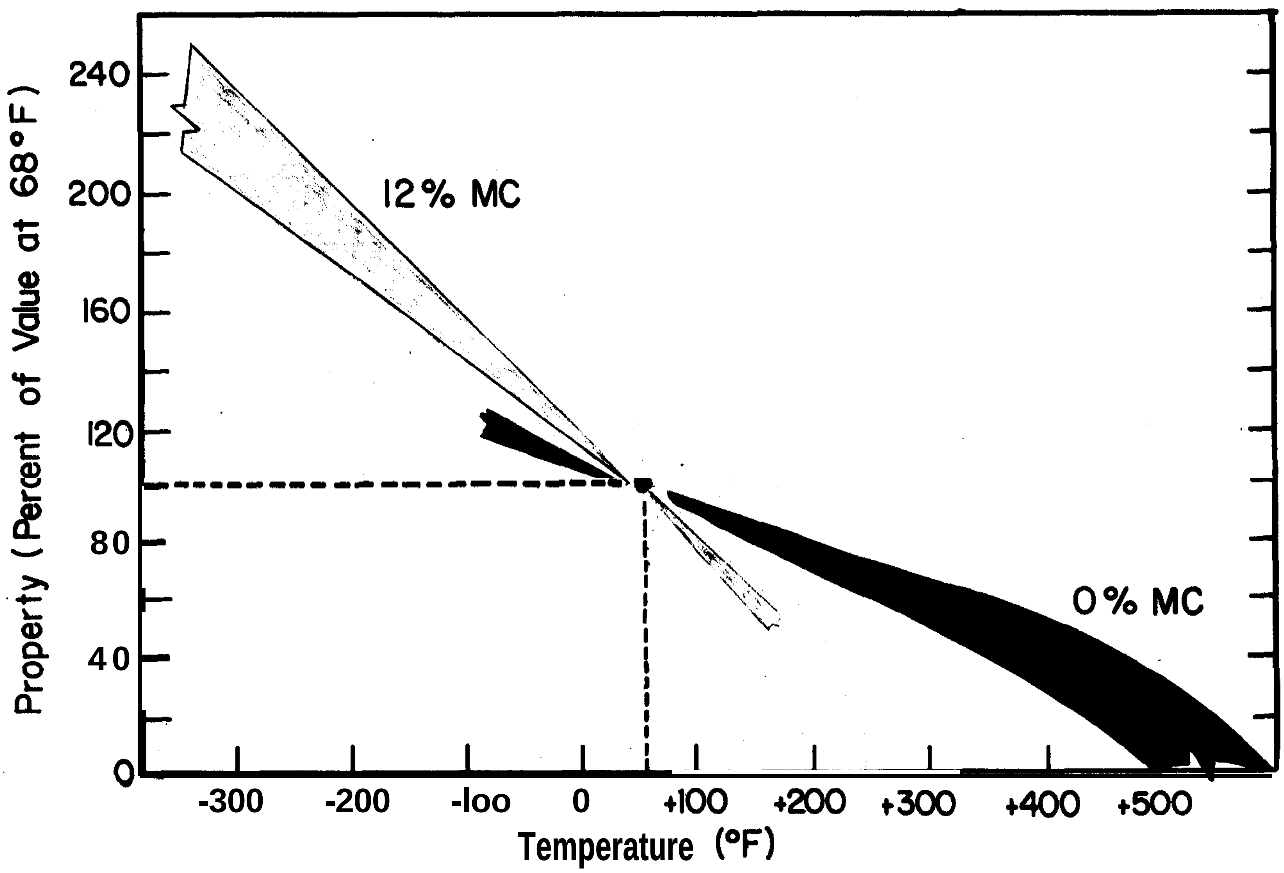


Figure 21. The Immediate Effect of Temperature. on Strength Properties, Expressed as Percent of Value at 68°F. Trends Illustrated are Composites From Studies on Three Strength Properties-Modulus of Rupture in Bending, Tensile Strength Perpendicular to Grain and Compressive Strength Parallel to Groin-as Examined by Several Investigators. Variability in Reported Results is Illustrated by the Width of the Bands.
(After the Wood Handbook, 1974)

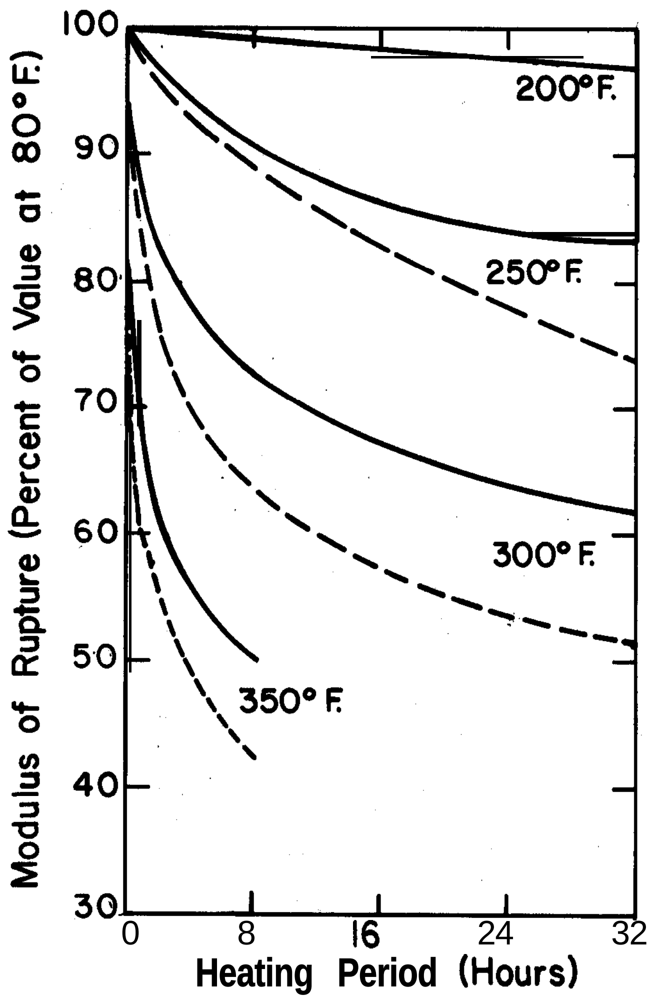


Figure 22– **Permanent Effect of Heating in Water (Solid Line) and in Steam** (Dashed Line) on the Modulus of Rupture. Data Based on Tests of Douglas-Fir **and Sitka** Spruce. (After the Wood Handbook, 1974)

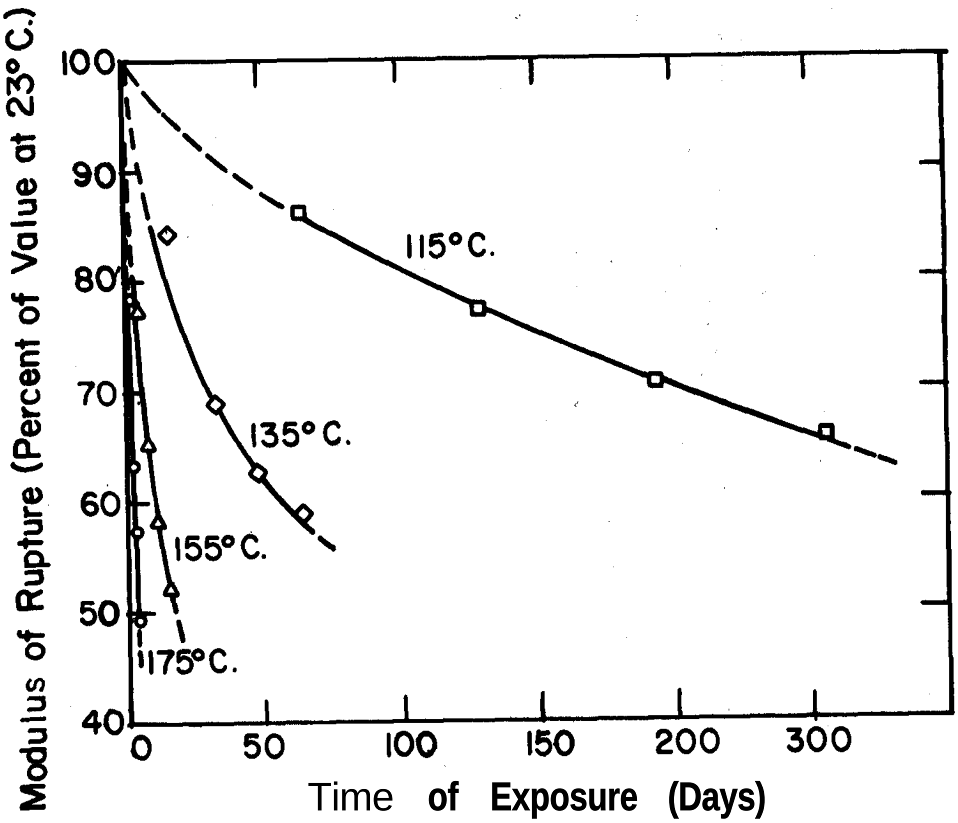


Figure 23 – **Permanent Effect of Oven Heating at Four Temperatures an the Modulus of Rupture, Bosed on Four Softwood and Two Hardwood Species. ('After Wood Handbook, 1974)**

applied rapidly so that failure is reached in approximately 1 second, the failure load would be approximately 25 percent higher than observed at the standard testing rate, On the other hand, if a specimen was subjected to a continuous load of only 60 percent of the failure load at the standard testing rate, rupture or failure of the specimen would not occur until approximately ten years after the sustained load was applied. This influence of load duration has been observed in all properties, and applies to both dry (6 to 12 percent moisture) and green wood (Markwardt, 1943; Wood, 1951; Wood, et. al. 1957; Sugiyama, 1967).

The relationship between failure stress and load duration most frequently referenced in the literature and used in wood design is presented in Figure 24. This relationship is a general trend line based on 126 long-term tests on 1 in, x 1 in. (25 mm x 25mm) clear Douglas fir beams at 6 and 12 percent moisture content, conducted at the U. S. Forest Products Laboratory at the University of Wisconsin (Wood, 1951). The specimens were\ subjected to constant loads ranging from 60 to 95 percent of the failure load for the standard loading rate (5 min.), and the specimens failed after load durations ranging from a few minutes to slightly in excess of 5 years. Data at loading rates faster than the standard rate for a similar series of tests are reported by Lislca (1950). The results of Liska's tests and the indicated scatter is shown in Figure 25.

The time required to obtain data on the load duration effect, similar to that on which Figure 24 was based, has led other researchers to study the phenomenon by performing creep tests (Sugiyama, 1967) where the strain-time relations at various stress levels are observed. The results of typical tests are shown in Figure 26 for bending and in Figure 27 for compression parallel to grain. The deformation-time curves in Figures 26 and 27 show the various portions of the classical creep curve consisting of: (1) the instantaneous elastic portion, (2) the initial curved portion which represents transient creep, (3) the straight line portion representing steady-state creep? and (4) the accelerating failure creep. Note that accelerated failure creep is evident only at stress levels greater than 50 percent of the failure stress at standard testing rates. Based on creep tests one might hypothesize that there exists some small stress level which can be carried indefinitely, under which the steady-state creep rate is zero. This limiting stress is referred to as the creep limit. In Table 12 the results of numerous investigations on creep properties are presented, as summarized by Sugiyama (1967). The data in Table 12 indicates that if a creep limit exists below which failure under long-term load is not a consideration, it is probably at a stress level of 40 to 50 percent (or lower) of the failure stress at standard testing rates.

The majority of the creep or load duration tests have been conducted at moisture contents of 6 to 12 percent. Limited data on green wood indicates that the creep limit can be greatly reduced at high moisture contents (Sugiyama, 1967 and Wood Handbook, 1974); hence, the use of Figure 24 might be somewhat unconservative for green timber. The creep properties of wood, like other materials exhibiting significant creep properties, are also highly influenced by temperature. The Wood Handbook, 1974, suggests that an "increase of 50°F (28°C) in temperature can cause a two-to-three-fold increase in creep." The effect temperature has on the creep limit is not known, but the creep limit probably decreases as the temperature increases.

In this section the general influences of moisture content,

TABLE 12 - SURVEY ON CREEP LIMIT VALUES BY OTHER AUTHORS

Prediction	Authors	Environmental conditions	Loading time	Creep Limit %	Remarks
Creep rupture	Markwardt	--	1 year	70	Predicted from experimental curve
	U.S. Forest Product Laboratory	controlled	5 years	60	No tests below 60
	Roth	--	--	55	--
	Takeyama		1 1/2 yrs.	40...50**	--
	Thurston	not controlled	15 months	67	No tests below 67
	Mori	--	3...9 months	40...50**	No tests below 40
	Hisada	--	4 months	50...60	--
	Graf	--	3 months	60	--
by rate of creep or creep function	Hrsada	not controlled	4 months	40	--
	Sugiyama	controlled	200..300 hrs.	40...50	About 40% in 2 tests series
	Sawada	controlled	300 hrs.	40	About 50% in other tests

* The creep limit is represented by load ratio or strain ratio.
** The Author assumes that the proportional limit in static tests is about 2/3 of the modulus rupture.

After Sugiyama (1967)

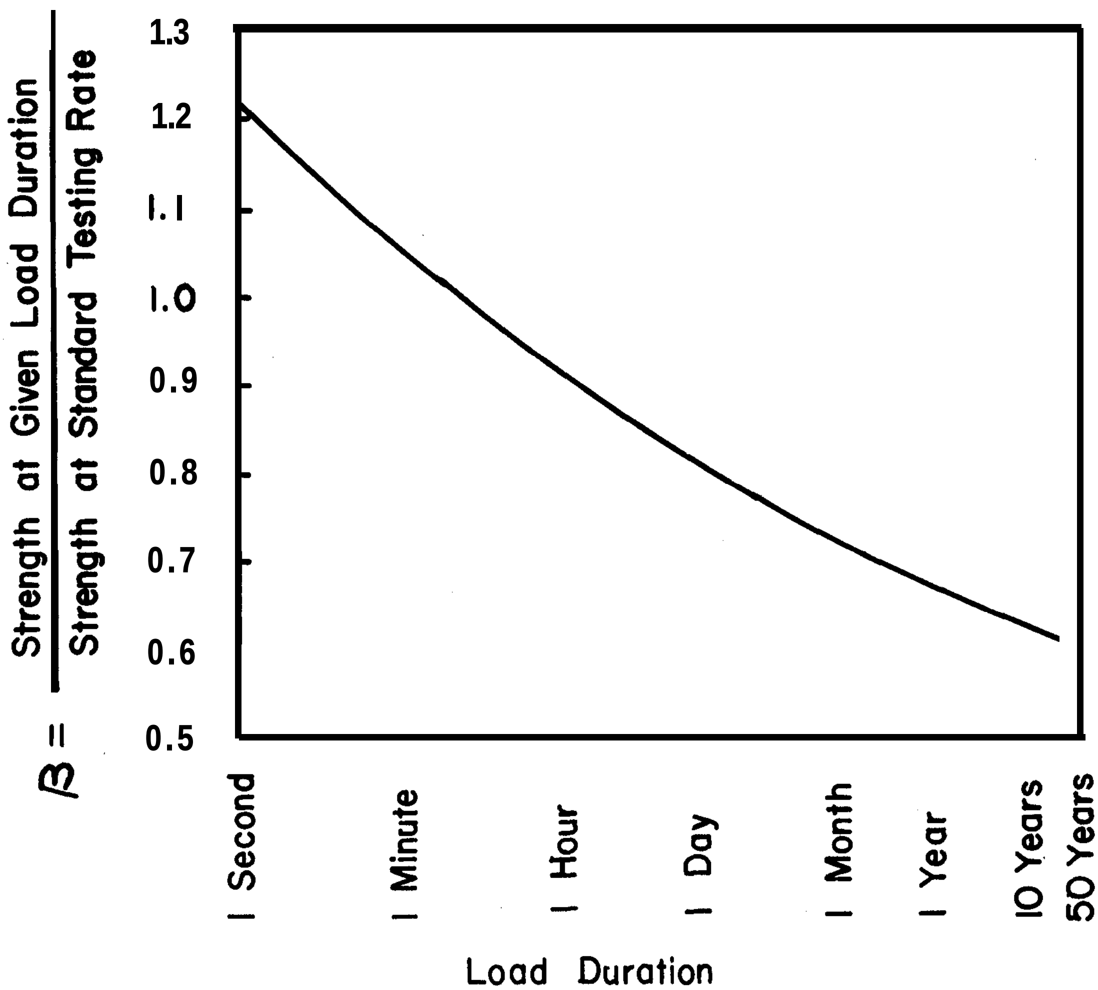


Figure 24: influence of Load Duration On Strength (After ASTM D 2555-76)

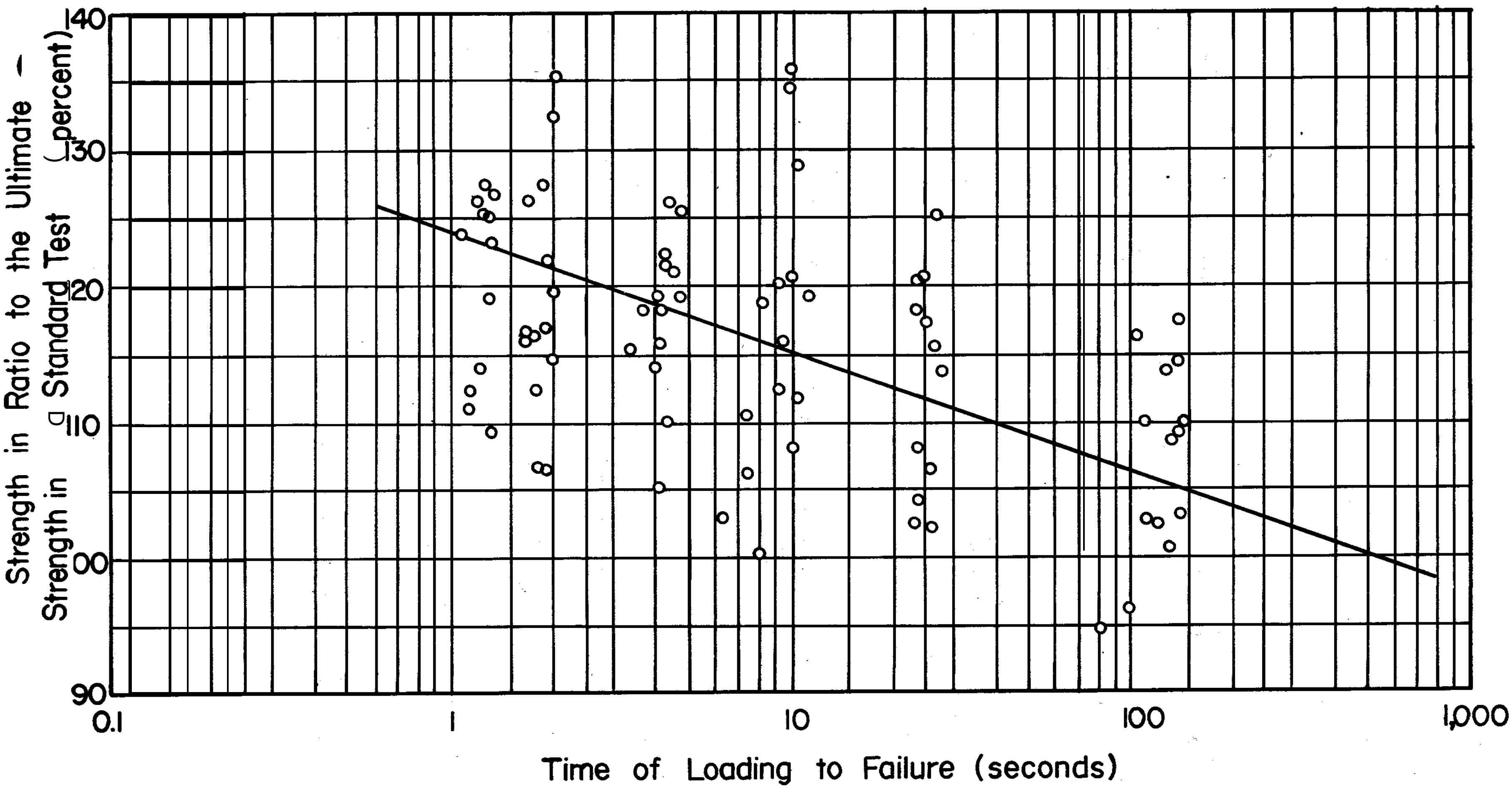


Figure 25: Relation of Strength to Time of Loading in Rapid-loading Tests of Small, Clear Douglas-fir Bending Specimens. (After Wood, 1951)

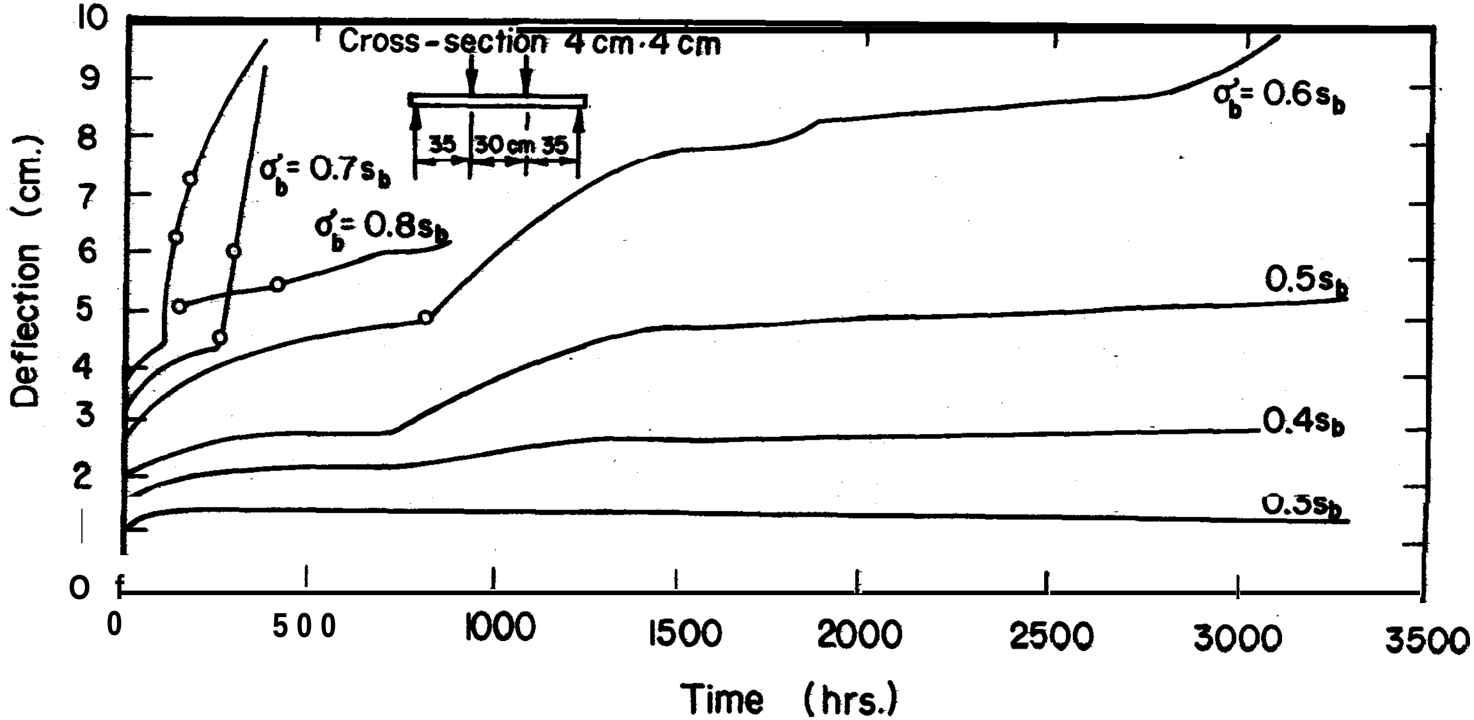


Figure 26 : Relation Between Time And Deflection In Creep Tests Under Bending Load.
(According to Hisada , 1950 /From Sugiyama , 1967)

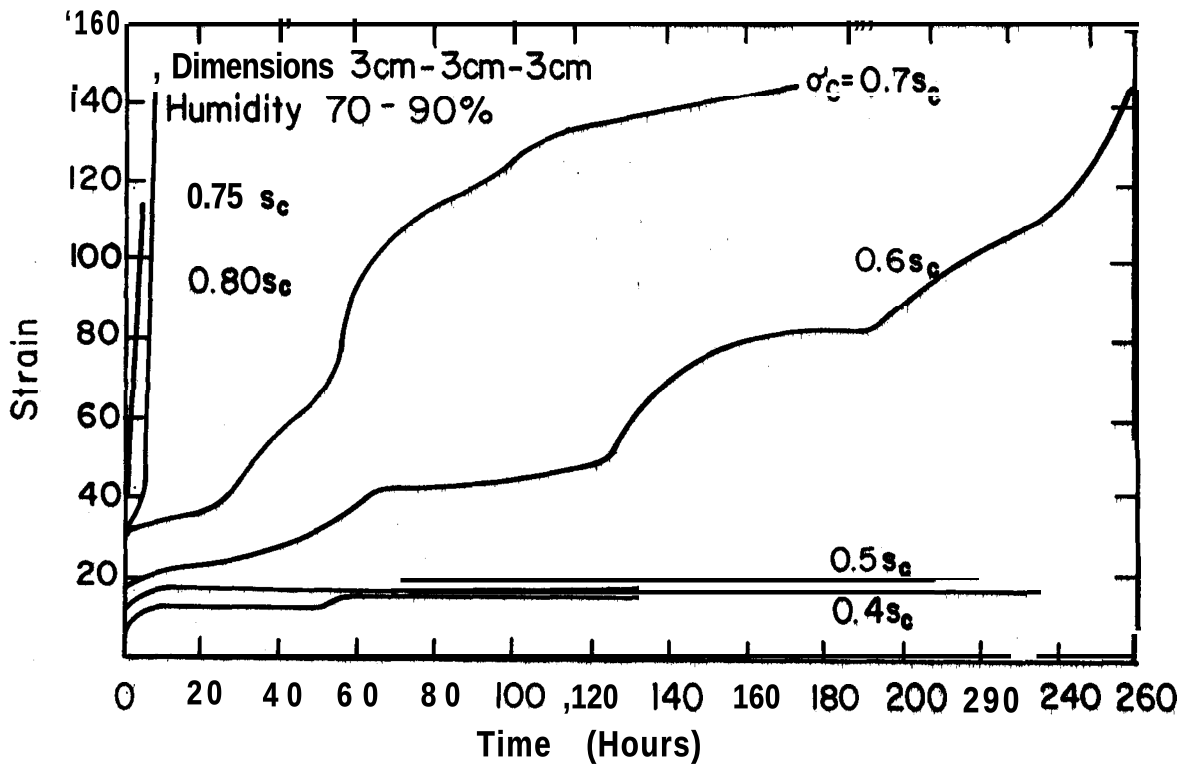


Figure 27 - Relation Between Time and Strain in Creep Tests Under Compression Load. (According to Hisada, 1950). (After Sugiyama, 1967).

temperature, and load duration on the strength of small clear wood specimens have been discussed, The large effect of these variables clearly indicates that the green small clear wood properties, such as tabulated in ASTM D2555, cannot be used directly in design without adjusting for the service environment (moisture, temperature, and load duration) under which the wood is utilized.

INFLUENCE OF IMPERFECTIONS ON WOOD STRENGTH

The preceding discussion *was* limited to clear wood specimens, free of all imperfections and defects. With respect to the structural use of wood, the assumption of clear wood represents a very idealized case. All trees contain natural growth imperfections (such as, spiral grain, knots, shakes, and splits), and additional defects (such as checks) can be introduced when the logs or lumber are conditioned.

Slope of Grain - The anisotropic nature of wood properties with respect to grain direction is illustrated in Table 6 where the compressive strength perpendicular to grain is shown to be only approximately 10 percent of the compressive strength parallel to grain. The compressive strength at some intermediate angle, θ, with respect to the direction of grain can be approximated by the Hankinson formula (National Forest Products Association, 1977).

$$F_n = F_c F_p/(F_c \sin^2 \theta + F_p \cos^2 \theta) \qquad \text{EQ.6}$$

where F_c = strength in compression parallel to grain
F_p = strength in compression perpendicular to grain
F_n = strength at an inclination θ with respect to the direction of grain

The Wood Handbook, 1974, presents a modified form of the Hankinson formula:

$$F_n = F_c F_p/(F_c \sin^n \theta + F_p \cos^n \theta) \qquad \text{EQ.7}$$

where n is an empirically determined constant. The Wood Handbook (1974) tabulates values of the constant n and values for the ratio of the strength property perpendicular to grain to the strength property parallel to grain (F_p/F_c), based on available literature, as follows:

Property	n	F_p/F_c
Tensile Strength	1.5 to 2.0	0.04 to 0.07
Compressive Strength	2.0 to 2.5	0.03 to 0.40
Rending Strength	1.5 to 2.0	0.04 to 0.10
Modulus of Elasticity	2.0	0.04 to 0.12
Toughness	1.5 to 2.0	0.06 to 0.10

Knots - A knot represents that portion of a branch that has been incorporated into the stem or trunk of a tree during the growth process. The knot itself will generally be composed of denser wood than the surrounding wood, and if tested by itself would likely exhibit a higher strength. However, the grain of the wood around the knot is distorted, and with larger knots this distortion can result in extremely steep slopes of grain. Hence, although the knot material may be stronger, knots decrease most strength properties of wood because of the distortion of grain and stress concentrations that are associated with their presence.

The presence of knots has a serious effect on the tensile strength of wood, and leas influence on compression parallel to grain. Because bending results in both tension and compression, the influence depends on the location of the knot within the member. The Wood Handbook (1974) indicates that ". . .in long columns knots are important in that they affect stiffness. In short or intermediate columns, the reduction in strength caused by knots is approximately proportional to the size of the knot; however, large knots have a somewhat greater relative effect than do small knots."

There is, in general, a lack of experimental data defining the strength reducing influence of knots. In the absence of such data, strength reductions applied in sawn lumber design are based on the theoretical assumption that the strength ratio (denoted herein by the symbol ϕ) is equal to the critical property of the cross-section assuming the knot material is absent to the property of the gross cross-section. Examples of these are given in the following equations for a rectangular section.

For compression parallel to grain:

$$\phi = \frac{\text{Area without knot}}{\text{Gross area}} = (1 - k/h) \qquad \text{EQ. 8}$$

For bending with knot on wide edge:

$$\phi = \frac{(I/c) \text{ without knot}}{(I/c) \text{ Gross area}} = (1 - k/h)2 \qquad \text{EQ. 9}$$

For bending with knot on narrow face:

$$\phi = \frac{(I/c) \text{ without knot}}{(I/c) \text{ Gross area}} = (1 - k/b) \qquad \text{EQ.10}$$

where b = width of narrow face
h = width of wide face
k = knot diameter
I/c = section modulus

Equation 8 is used for both the strength reduction for compression parallel to grain, and the strength reduction for bending with knots along the centerline of the wide face. Equation 9 is used for bending with knots on the edge of the wide face, and Equation 10 is used for bending with knots on the narrow face. The strength ratios defined by Equations 8 through 10 are shown on Figure 28 as a function of the dimensionless knot ratios, k/h and k/b.

Shakes; Checks and Splits - The presence of shakes reduces the area available to resist horizontal shear in a member subjected to bending, but has little or no influence on compression parallel to grain. Checks and splits are treated in timber design as equivalent shakes when evaluating their strength reducing effects. The strength ratio for horizontal shear used in current design practice is approximated by Equation 11, where w is the check width and b is the narrow face width of the beam.

$$\phi = 1 - w/b \qquad \text{EQ. 11}$$

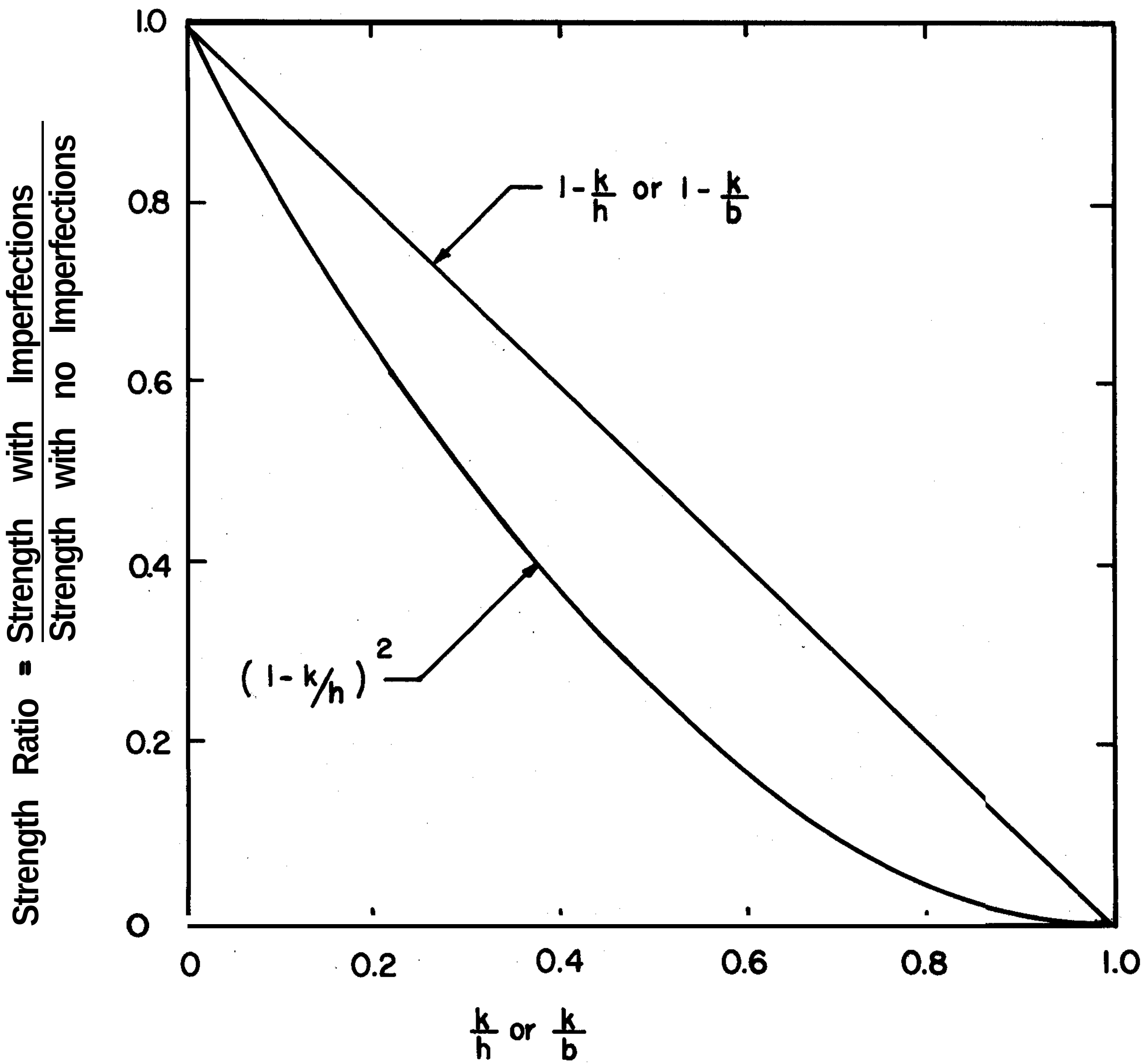


Figure 28. Strength Ratios From Equations 8-10

DESIGN OF SAWN LUMBER

In the preceding sections, a general discussion was presented of the large variability of clear wood strength, factors influencing clear wood strength, and the influence of natural growth imperfections on strength. The method by which these complex variables are considered in developing allowable stresses is illustrated in this section by reviewing current design practices in sawn lumber.

Allowable stresses in sawn lumber are based on ASTM Standard Methods for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber (D245-74), and ASTM Standard Method for Evaluating Allowable Properties for Grades of Structural Lumber (D2915-74). The general design philosophy used in both these methods is the same, although the method of evaluating the influence of the various factors is different. In ASTM D245, the lumber is sorted into classes according to visual observation of imperfections, and a strength ratio is assigned to each class based on theoretically or experimentally developed formulas. The strength ratio, ϕ, is selected as the smallest strength ratio determined for the various imperfections permitted in the class or grade of lumber. The five percent exclusion property for green small clear specimens, s', is determined from the information provided in ASTM D2555 as described in the preceeding section on variations in clear wood strength. In ASTM D2915, lumber is sorted into classes and the strength ratios associated with each class or grade are assessed by statistical analysis of actual test data on a representative sample of the class. Both methods are based on assigning a value to the property that represents a 5 percent exclusion limit for the grade or class at standard testing rates. This value is then converted to an allowable wood property for in-use conditions by applying adjustment factors to the property that accounts for duration of load, a minimum factor of safety, moisture content, temperature, and where necessary, size effects.

A detailed discussion of the grading processes for sawn lumber, or the procedures whereby sawn lumber is sorted into classes containing different degrees of imperfections or density, are beyond the scope of this report. In general, the classes or grades of sawn lumber commercially available are controlled by trade organizations. For more detailed information on the grading process and the use of allowable stresses in sawn lumber, numerous publications are available (Gurfinkel, 1981 and the Wood Handbook, 1974). In the following presentation, only the method for visually graded lumber (ASTM D245) is discussed.

The allowable property of green (unseasoned) lumber used as a structural member under normal load duration and normal temperature conditions, s_{ag}, can be expressed by the following formula:

$$s_{ag} = f\phi s'/F_{sd} \qquad \text{EQ. 12}$$

In Equation 12 the strength ratio, ϕ, represents a reduction accounting for the presence of imperfections; the adjustment factor, f, accounts for the influence of size and shape on member strength; and the adjustment factor, F_{sd}, is a composite adjustment for normal load duration and a near minimum factor of safety. These adjustment factors are applied to the 5 percent exclusion property for green small clear wood, s', to obtain an allowable green property (allowable property in green or unseasoned condition) for the member under normal load duration and temperature conditions, 8, .

The allowa% le green property for normal load duration, Sag, must be modified for design use to account for the various factor8 that can influence wood Strength under actual service conditions. The three major use modifications are moisture content, load duration, and temperature. The allowable design property adjusted for these factors, Sad, can be expressed by the following formula:

$$s_{ad} = m\ \psi\ (\ \beta\ /0.625) s_{ag} \qquad \text{EQ. 13}$$

where the adjustment factor, m, modifies the property to the in-use moisture content; the adjustment factor, ψ, modifies the property to the in-use temperature; and the adjustment factor, β, modifies the property to the in-use load duration. The 0.625 in the denominator of Equation 13 is the β-value for normal load duration, and must be included because the al lowable green property , s_{ag}, was already adjusted to normal load duration in Equation 12.

In the next Section, where timber piles will be considered, it will be useful to combine equation8 12 and 13 and modify the notation to the following form:

$$s_{ad} = f\ m\ \psi\ \beta\ \phi\ s'/f_s \qquad \text{EQ. 14}$$

Where m, f, ψ , β , and ϕ are the adjustment factor8 previously defined; s' is the 5 percent exclusion small clear wood Strength value; and f_s is the formal factor of safety included to obtain an allowable stress. The value of f_s for sawn lumber is given by:

$$f_s = 0.625 F_{sd} \qquad \text{EQ. 15}$$

where F_{sd} is the combined adjustment for normal load duration and factor of safety. A discussion of the values of the various adjustment factor8 used in Equation 12 through 15 for sawn lumber design follows.

Adjustment for Moisture Content, m - The adjustment factor for moisture content, m, accounts for the increase in wood strength that occurs when the moisture content is reduced below the fiber saturation point. ASTM D245 gives adjustment factors at 15 and 19 percent moisture based on the relationship given in Equation 5 with additional restrictions on the dimensions. The increase in strength associated with moisture content is of ten of feet by shrinkage and seasoning defects that occur during the drying process for pieces larger than approximately 4 inches (102 mm) (Wood Handbook, 1974). hence, ASTM D245 restricts consideration of the strength increase associated with moisture content to lumber 4 inches (102 mm) or less in nominal thickness.

Ad justment for Size and Shape, f - The bending strength of a member is dependent on both the size and shape of the member (Wood Handbook, 1974; Gurfinkel, 1981; and NFPA, 1977). The bending strength, as indicated by the modulus of rupture, ha8 been found to decrease as the depth of the section increases (Bohannan, 1966). For a rectangular member the form or shape factor is 1.0, and the adjustment factor for size and shape is based.

on Equation 16, where d is the net surfaced depth in inches.

$$f = (2/d)^{1/9} \qquad \text{EQ. 16}$$

When the cross-sectional shape of a beam is other than rectangular, an adjustment for shape is required, and Equation 16 must be modified to the following form:

$$f = a(2/d)^{1/9} \qquad \text{EQ. 17}$$

where a is the form or shape factor. For round members the modulus of rupture is approximately 1.18 times the modulus of rupture for rectangular members; hence, the form factor is 1.18 for round members. For a square member subjected to bending about one of its diagonals, the form or shape factor is 1.414 (NFPA, 1977). The adjustment factor for size and shape, f, is equal to 1.0 for all properties except the bending strength.

Adjustment for Temperature, ψ - The adjustment factor for temperature accounts for both the immediate and permanent effect of temperature on wood strength when subjected to other than normal temperature. In the environment under which wood is normally used, the immediate effect of temperature is not generally significant. However, if wood is being considered for use in a high temperature environment for an extended period of time, "immediate" and "permanent" effect of temperature must be considered. ASTM D245 does not provide specific recommendations on values for the adjustment factor for temperatures above normal. The National Design Standard for Wood Construction (NFPA, 1977) provides some guide lines for making adjustments for in-use temperatures when wood members are cooled to very low temperatures or heated to 1500F (680C) for extended periods of time.

The permanent strength loss can be a serious design consideration when dealing with treated timber. The high temperature and pressure used for conditioning of wood at a high moisture content under approved methods of treatment have been demonstrated by test to cause permanent strength losses. While the strength loss caused by the conditioning process can be minimized by restricting the conditioning temperatures, heating periods, and pressures, the restrictions must be consistent with the primary goals of absorption and penetration as required for proper treatment.

Adjustment for Imperfections, ϕ - In visually graded lumber (ASTM D245) the adjustment factor for imperfections is referred to as the strength ratio for the grade. This strength ratio is defined as the ratio of the strength of a member containing the maximum permitted imperfections in the grade to the strength of a member free of imperfections. A given grade would have permissable limits on the various imperfections such as: knots, slope of grain, shakes, checks, and splits. Each of these imperfections may have an influence on a particular property, and each must be considered. However, the strength ratio that is used to derive the strength property for the grade is the lowest ratio determined for the various defects.

For example; consider the strength ratios for a 4 inch (102 mn) by 8 inch (203 mm) member of a grade that permits knots up to 3/4 inch (19.1 mm.) diameter in the narrow face, knots up to 1/2 inch (12.7 mm.) in the edge of the wide face, knots up to 3/4 inch (19.1 mm) along the center of the wide face, and slopes of grain up to 1:15. The strength reducing effects of these imperfections, are shown in Table 13 based on: (1) the various equations in the preceeding section on imperfections, and (2) ASTM D245 recommendations. The bending strength ratios are presented in Table 13a and the compression strength ratios in Table 13b. Based on the formulas presented in the preceeding section, the strength ratio applied to the modulus of rupture for this example is 0.81, and is controlled by knot dimensions in the narrow face. In compression parallel to grain, the strength ratio would be 0.91, and is controlled by the largest knot.

The strength ratios for the knot limitations in ASTM D245 are slightly higher (1 to 3.5 percent) than those given by equations 8 through 10 because of the slightly more complex form of the ASTM D245 equations for strength ratios. For example, the ASTM D245 equation for the bending strength ratio associated with knots in a narrow face of less than 6 inch width, and for strength ratios less than 45 percent, is given in Equation 18, where k is the knot diameter in inches and b is the width of the narrow face in inches.

$$\phi = 1-(k-1/24)/(b + 3/8) \qquad \text{EQ. 18}$$

In lumber, several knots within close, proximity of each other could be more critical than a single knot of larger size. ASTM D245 limits the sum of knot diameters in a specified length, dependent on the use of the member, to a multiple of the permitted maximum knot size. For example, in joists, planks, posts, and timbers; the sum of knots in any 6 inch (152 mm) length of the member may not exceed twice the size of the largest permitted knot diameter. Hence, the strength reduction for the sum of knots in 6 inches (152 mm) can be estimated by Equations 8 through 10 by using an effective knot diameter, k', equal to one-half the sum of knots.

The ASTM D245 reduction for slope of grain in the above example is approximately 80 percent of the value estimated on the basis of the conventional Hankinson formula (Equation 6). The strength ratio for the modulus of rupture, as determined by tests with various slopes of grain, is presented in Table 14 (Wood Handbook, 1974) and these data points are shown in Figure 29 by the solid circular symbols. The solid lines in Figure 29 *are* theoretical curves based on a Hankinson-type formula of the following form:

$$\phi = R/[\sin^{1.62}W + R\cos^{1.62}W] \qquad \text{EQ.19}$$

Where R is the ratio of the strength perpendicular to grain to the strength parallel to grain. Curve A is based on a value of 0.1 for R, and W is the slope of grain θ . Curve B is also based on a value of 0.1 for R, but the angle W is taken as twice the slope of grain, 20. The ASTM D245 strength ratios for slope of grain are presented in Table 15 and are shown on Figure 29 by dashed lines for compression parallel to grain and for the bending strength. Curve A in Figure 29 shows an extremely. good fit to the experimental data in bending.

The Wood Handbook (1974) indicates that in order to provide a margin of safety, the reduction in strength due to **cross** grain in visually graded structural lumber should be about twice the reduction observed in tests of small clear specimens that contain similar *cross* grain.. The reasoning is

TABLE 13 - EXAMPLE CALCULATION OF STRENGTH RATIO FOR IMPERFECTIONS

Member Dimensions: 4 inch by 8 inch (10.2 mm by 20.3 mm)
Permitted Imperfections:
- slope of grain 1:15
- knots in narrow face up to 3/4 inch (19 mm)
- knots in edge of wide face up to 1/2 inch (13 mm)
- knots along center of wide face up to 3/4 inch (19 mm)

TABLE 13a - Strength Ratio in Bending

Imperfection	Strength Ratio	Equation	ASTM D245 Strength Ratio
Knots in Narrow Face		10	0.84
Knots in edge of Wide Face	0.88	9	0.90
Knots at center of Wide Face	0.91	8	0.92
Slope of grain	0.96(a)	6	0.76

(a) assumes ratio of strength parallel to grain to strength perpendicular to grain is 10.0

TABLE 13b - Strength Ratio in Compression Parallel to Grain

Imperfection	Strength Ratio	Equation	ASTM D245
Knots	0.91	8	0.92
Slope of grain	0.96	6	1.0

TABLE 14 - STRENGTH OF WOOD MEMBERS WITH VARIOUS GRAIN SLOPES COMPARED TO STRENGTH OF A STRAIGHT-GRAINED MEMBER

Maximum Slope of grain in member	Modulus of Rupture	Impact bending - height of drop causing complete failure (50-lb. hammer)	Compression parallel to grain - max. crushing strength
	Pct.	Pct.	Pct.
Straight-grained - - -	100	100	100
1 in 25 - - - - - - - -	96	95	100
1 in 20 - - - - - - - -	93	90	10C
1 in 15 - - - - - - - -	a9	al	10C
1 in 10 - - - - - - - -	81	62	99
1 in 5 - - - - - - - - -	55	36	93

From Wood Handbook (1974)

TABLE 15. - ASTM D245: STRENGTH RATIOS CORRESPONDING TO VARIOUS SLOPES OF GRAIN

Slope of Grain	Maximum Strength Ratio, percent	
	Bending or Tension Parallel to Grain	Compression Parallel to Grain
1 in 6	40	56
1 in a	53	66
1 in 10	61	74
1 in 12	69	a2
1 in 14	74	a7
1 in 15	76	100
1 in 16	80	
1 in la	a5	
1 in 20	100	

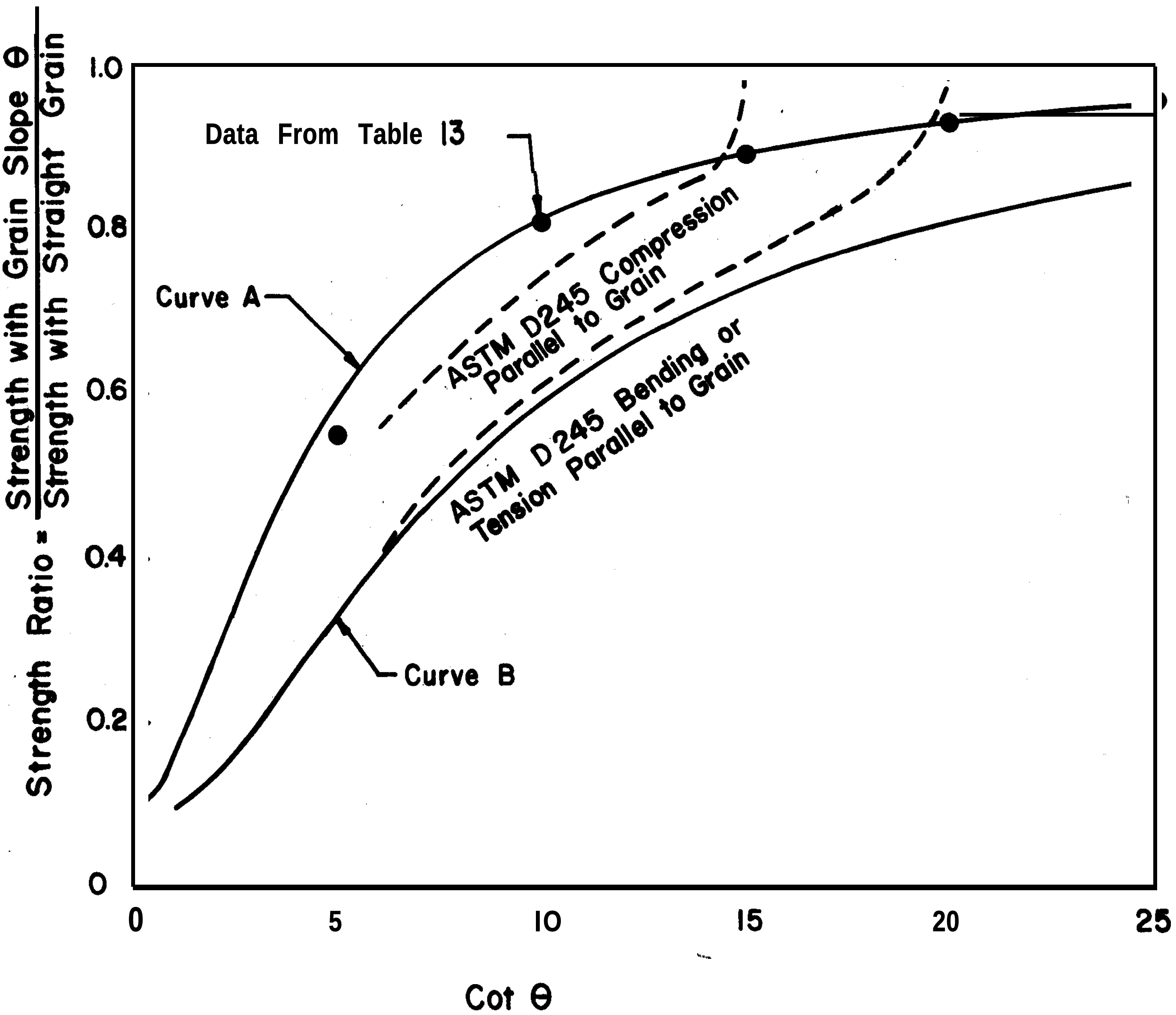


Figure 29. Strength Ratio for Slope of Grain.

that the shrinkage stresses and warping during drying are greater in structural lumber than in small clear specimens. It appears that ASTM D245 includes this recommendation because, for slopes greater than **1:16,** the D245 strength ratio for bending closely follows curve B which is based on **2θ.** Although not shown in Figure 29, the ASTM **D245** strength ratio for sloping grain in compression parallel to grain is closely approximated by Equation 19 using **$R=0.2$** and **$W=2\theta$,** for grain slopes greater than **1:14.**

Adjustment for Load Duration, β, and Factor of Safety, f_s - The adjustment factor, F_{sd}, in Equation 12 is a composite adjustment for normal duration of load and a factor of safety. Normal load duration is a commonly used term in timber design, which contemplates fully stressing a member to the allowable stress by the application of the full maximum design load for a duration of approximately 10 years, either continuously or cumulatively , or the **application** of 90 percent of this full maximum load continuously throughout the remainder of the life of the structures, or both, without encroaching on the factor of safety (ASTM D245). The values for the adjustment factor, Fsd, recommended by ASTM D245 are shown in Table 16, where a distinction is made between softwoods and hardwoods. The **values** for hardwoods are 10 percent higher than for softwoods.

The basis of the load duration adjustment in sawn lumber is the relationship in Figure 24. At a load duration of 10 years the relationship in Figure 24 indicates a load duration factor,8 , of 0.625. If the loading is other than normal duration the allowable design stress can be adjusted by the relationship in Figure 24. The load durations usually used for various types of load are summarized in Table 17, along with the corresponding values of the adjustment factor, β (National Forest Products Association, 1977). Section 1.10.1 of the Standard Specification for Highway Bridges (AASHTO, 1977) provides similar values for load duration adjustment .

Based on a β -value of 0.625, the value of the factor of safety, f_s, in bending and compression parallel to grain included in the ASTM D245 adjustment factor F_{sd} is on the order of 1.2 to 1.4. A considerably larger factor of safety has been included for horizontal shear because of the uncertainty in calculating shear stresses in checked beams using elastic expressions (Gurf inkel, 1981).

Adjustment for Combined Loading - Although the primary load component for columns is usually axial load, columns are also frequently subjected to horizontal forces which result in bending moments. Also, columns are almost always subjected to some eccentricity of the axial force due to normal construction tolerances, initial curvature, and material nonhomogeneity. Hence, columns should be designed for some minimum eccentricity to account for bending stresses that may develop even though the idealized theoretical load components are computed to be concentric (Gurf inkel, 1981). The presence of combined axial thrust and bending in timber is addressed by straight line service load interaction formulas. For short columns (**kL/d** less than **11**), the interaction formula becomes (Gurfinkel, 1981 and National Forest Products Association, **1977**):

$$fc/F'c + fb/Fb' \leq 1 \qquad \text{EQ. 20}$$

where

fc = stress in compression parallel to grain induced by axial load

TABLE 16. ASTM D245 VALUES FOR THE ADJUSTMENT FACTOR, Fsd (1)

Strength Property	Adjustment Factor	
	Softwoods	Hardwoods
Modulus of Rupture	2.1	2.3
Compression Parallel	1.9	2.1
Horizontal Shear	4.1	4.5

(1) includes adjustment for normal load duration and a factor of safety.

TABLE 17. LOAD DURATIONS AND ADJUSTMENT FACTORS FOR VARIOUS TYPES OF LOADS (1)

Type of Loading	Duration	Adjustment Factor
Dead	Permanent	0.5625
Live	10 years	0.625
Snow	2 months	0.7188
Wind & Earthquake	1 day	0.8313
Impact	1 second	1.25

(1) adapted from the National Design Specification for Wood Construction, 1977.

f_b = flexural stress

F_c' = allowable design stress in compression parallel to grain that would be permitted if axial stress only existed

F_b' = allowable design stress in bending that would be permitted if flexural stress only existed

For longer columns proper design requires consideration of potential buckling of the unsupported portion and the P-6 effect. A comprehensive treatment of wood column behavior and design is presented by Gurfinkel (1981).

ALLOWABLE STRESSES FOR ROUND TIMBER PILES

In the preceding section the general approach used in developing allowable stresses in sawn lumber and the basis of the required adjustment factors was presented. A rational evaluation of allowable structural stresses for timber piles can follow this same approach, although the difference in behavior of round timber piles as compared to sawn lumber leads to changes in the various strength ratios and adjustment factors.

Timber piles are generally used in environments that would preclude consideration of moisture contents less than the fiber saturation point. Furthermore, since they are almost always larger than 4 inches (102 mm) in dimension, defects associated with seasoning would offset any strength gains from moisture reduction. Hence, the adjustment factor for moisture content should be 1.0. If it is also noted that the size adjustment factor in compression is 1.0, then equation 14 for the case of compression parallel to grain for a concentrically loaded column reduces to the expression for allowable compressive working stress for timber piles presented by Armstrong (1979).

Armstrong (1979) restricted his consideration to axial, compression without, associated bending stresses. In addition, al though references were made to the difference in strength behavior of timber pile tips and butts, Armstrong did not include a specific adjustment for this effect. In order to provide a complete discussion, these variables will be included here. The variation in strength from butt to tip can be accounted for by an adjustment factor denoted by the symbol, γ , and the influence of an accidental eccentricity of the axial load can be considered by using an adjustment factor denoted by the symbol, ϵ . In addition, a separate reduction factor, denoted by the symbol HDF, is incorporated to account for potentially unnoticed structural damage that occurs during pile driving. Incorporating these factors and using the subscripts, c and b to denote compression parallel to grain and bending respectively, the allowable design stresses for compression and bending are given by Equations 21 and 22.

$$s_{ac} = HDF(1/f_{sc})\ \epsilon\ \psi\ \gamma\ \beta\ \phi_c s_c' \qquad \text{EQ. 21}$$

$$s_{ab} = HDF(1/f_{sb})\ f\ \psi\ \gamma\ \beta\ \phi_b s_b' \qquad \text{EQ.22}$$

where:

s_a = allowable design stress

s' = 5 percent exclusion value for green small clears from ASTM D2555

β = adjustment factor for load duration

ψ = adjustment factor for above normal temperature conditions

ϕ = strength ratio associated with imperfections
γ = adjustment factor for difference in strength between pile tip and butt
f = adjustment factor for influence of size and shape on modulus of rupture
ε = adjustment for minimum design eccentricity in axial compression
HDF = adjustment factor for hidden defects
f_s = adjustment factor of safety required to obtain an allowable stress

Material standards and information from previous studies on strength of round timber members are summarized for the purpose of establishing appropriate design values for the various strength ratios and adjustment factors, consistent with the AASHTO pile material specifications and the use of timber piles in the highway industry. In some cases the available test data are not accompanied by sufficient supporting information, such as the extent of defects present or treatment details, to allow the influence of the individual variables to be isolated. Where the existing data are not sufficient to allow definitive values to be established, conservative values are assigned until additional information can be obtained.

Material Standards - Timber piles are purchased or supplied to a job to meet specification requirements such as dimensions, straightness, limitations on imperfections, and treatment requirements. In some cases the detailed requirements may be spelled out in the project specification. However, more frequently, the material requirements will be specified by referencing a standard material specification.

One of the most frequently used specifications for timber piles is the ASTM Standard Specification for Round Timber Piles (D25). The first version of ASTM D25 was issued in 1915, and the most current version was issued in 1979. Prior to 1970 (D25-58 and earlier versions) ASTM D25 classified timber piles into three general divisions according to the intended use, as follows:

Class A: Piles suitable for use in heavy railway bridges or other heavy framed construction.
Class B: Piles suitable for use in docks, wharves, bridges, building or other foundations, and general construction.
Class C: Piles suitable for use in foundations which will always be completely submerged, for cofferdams, falsework, or light construction.

The size requirements for various classes were different as indicated by the D25-58 requirements which are presented in Table 18. Other quality requirements such as straightness, knots, holes, splits and shakes were the same for both class A and class B piles. The imperfection limitations were not as stringent for Class C piles. The permissible slope of grain was the same for all classes. Limitations on knots were specified in terms of the largest knot diameter k_L, the sum of knot diameters in 1 ft (305 mm) pile length k_S, the ratio of the largest knot to the pile diameter k_L/D, and the ratio of the sum of knots in 1 ft. (305 mm) to the pile diameter k_S/D. In addition, D25-58 and earlier versions prohibited knot clusters.

In 1970 D25 underwent a major revision with minor revisions following

TABLE 18: ASTM D25-58 CIRCUMFERENCES AND DIAMETERS OF TIMBER PILES

Length, ft	Class A: 3 Ft from Butt, Min: Circumference, in.	Class A: 3 Ft from Butt, Min: Diameter (approx) in.	Class A: 3 Ft from Butt, Max: Circumference, in.	Class A: 3 Ft from Butt, Max: Diameter (approx) in.	Class A: At Tip, Min: Circumference, in.	Class A: At Tip, Min: Diameter (approx) in.	Class B: 3 Ft from Butt, Min: Circumference, in.	Class B: 3 Ft from Butt, Min: Diameter (approx) in.	Class B: 3 Ft from Butt, Max: Circumference, in.	Class B: 3 Ft from Butt, Max: Diameter (approx) in.	Class B: At Tip, Min: Circumference, in.	Class B: At Tip, Min: Diameter (approx) in.	Class C: 3 Ft from Butt, Min: Circumference, in.	Class C: 3 Ft from Butt, Min: Diameter (approx) in.	Class C: 3 Ft from Butt, Max: Circumference, in.	Class C: 3 Ft from Butt, Max: Diameter (approx) in.	Class C: At Tip, Min: Circumference, in.	Class C: At Tip, Min: Diameter (approx) in.
Douglas Fir, Hemlock, Larch, Pine, Spruce, or Tamarack							a	a					b	b				
Under 40	44	14	57	18	28	9	38	12	63	20	25	8	38	12	63	20	25	8
40 to 54 incl.	44	14	57	18	28	9	38	12	63	20	22	7	38	12	63	20	19	6
55 to 74 incl.	44	14	57	18	25	8	41	13	63	20	22	7	38	12	63	20	19	6
75 to 90 incl.	44	14	63	20	22	7	41	13	63	20	19	6	38	12	63	20	19	6
Over 90	44	14	63	20	19	6	41	13	63	20	16	5	38	12	63	20	16	5
Oak and Other Hardwoods; Cypress							a	a					b	b				
Under 30	44	14	57	18	28	9	38	12	57	18	25	8	38	12	63	20	25	8
30 to 40 incl.	44	14	57	18	28	9	41	13	63	20	22	7	38	12	63	20	22	7
Over 40	44	14	57	18	25	8	41	13	63	20	19	6	38	12	63	20	19	6
Cedar							a	a					b	b				
Under 30	44	14	69	22	28	9	38	12	69	22	25	8	38	12	69	22	25	8
30 to 40 incl.	44	14	69	22	28	9	41	13	69	22	25	8	38	12	69	22	25	8
Over 40	44	14	69	22	25	8	41	13	69	22	22	7	38	12	69	22	22	7

(a) In Class B piles, a minimum circumference of 34 in. or diameter of 11 in. at a point 3 ft. from the butt may be specified for lengths of 25 ft and under.

(b) In Class C, a minimum circumference of 31 in. or diameter of 10 in. at a point 3 ft from the butt may be specified for lengths of 25 ft and under.

Note: 1 in. = 25.4 mm; 1 ft. = 305 mm

in 1973. The three class system was dropped, with all piles being lumped into one class. The limitations on straightness, holes, splits and shakes were those previously specified for Class A and B piles. However, the knot limitations were relaxed such that piles which would not meet the liberal requirements of the former class C piles now became the standard. The revision in the knot limitations of D-25-70 is illustrated in Table 19 where they are compared to the D25-58 limitations. In 1979 D25 was revised to return the permissible sum of knots to the value previously permitted for class C piles. Hence, with the exception of the specific limitations on maximum knot size k_L, the maximum sum of knots k_S, and the now permitted cluster knots, the current D25-79 piles are equivalent to the previous (D25-58) class C piles for lengths less than 50 ft (15.2 m); for lengths greater than 50 ft they are approximately equivalent to the previous (D25-58) class A and B piles.

The 1970 revision of D25 also introduced a change in the manner in which the pile dimensions are specified. Under the current standard, D25-79, the designer can specify either the minimum butt circumference, Table 20a, or the minimum tip circumference, Table 20b. Once the minimum circumference at one end is specified, the minimum circumference at the other end is determinable from Table 20.

Several organizations other than ASTM have their own specifications for pile quality. However, a close review of such standards will generally indicate that they were adapted from some version of D25. While some organizations adopt the most current version of D25 without review, other organizations are either slower, or more cautious, when adopting revisions. For instance the American Standards Association has methodically adopted the current version of D25 within a year or two after a revision is made (ASA No. 061). On the other hand, for some organizations, the current specification for wood piles is primarily the 1937 version of D25. For example, the current AREA (American Railway Engineering Association) specification for wood piles is almost identical to D25-37. The 1970 revision of D-25 has received only token acceptance by industry and engineers, although it has been adopted by the major national building codes. Some local building codes are reluctant to adopt the latest revision of D25 because of the extremely lenient knot limitations (Dwyer, 1975).

The current AASHTO Standard Specification for Structural Timber, Lumber, and Piling (AASHTO Designation: M 168-65 (1974)) has provisions traceable to D25-37 and D25-58. The AASHTO Standard Specification provides for only one class or grade of pile. However, a comparison of the AASHTO M-168 size and imperfection limitations indicate that the single class is identical to the pre-1970 D25 class B pile. Considering D25-37 through D25-58 nomenclature, the use of timber piles in the highway industry falls under Class B. Hence, the use of only one class in AASHTO M-168 is not a deviation from the earlier versions of D25.

Since the major goal of this chapter is to develop allowable stresses in timber piles for highway use, the detailed requirements of AASHTO M-168 are of concern because the specification limitations on imperfections can influence pile strength. The resulting allowable unit stress combined with the minimum allowable dimension controls the allowable loads that can be developed. The current AASHTO dimension limitations are reproduced in Table 21. Comparison of Table 21 with the D25-58 size limitations for

TABLE 19 - COMPARISON OF KNOT LIMITATIONS FOR ASTM D 25-58 AND ASTM D 25-70

KNOT LIMITATION	ASTM D 25-58 Class A and B Piles[a][c]	ASTM D 25-58 Class C Piles[c]	ASTM D 25-70 & D 25-73
k_L	4 inches	5 inches	No limitation
k_S	8 inches	10 inches	No limitation
k_L/D	1/3	1/3	1/2
k_S/D	2/3	1	2.09
Cluster Knots	Not permitted	Not permitted	(b)

(a) Knot limitations quoted for Class A and B piles are for lengths less than 50 feet (15.2 m). For lengths greater than 50 ft (15.2 m), the knot limitations for class A & B apply to the upper three-quarters of the pile length and the limitations of class C apply to the lower one-quarter of the pile.

(b) Cluster knots are permitted. Cluster knots shall be considered as a single knot, and the entire cluster cannot be greater in size than permitted for a single knot.

(c) 1 in. = 25.4 mm.

TABLE 20 - ASTM D25-79 CIRCUMFERENCES AND DIAMETERS OF TIMBER PILES

20a: SPECIFIED BUTT CIRCUMFERENCES WITH MINIMUM TIP CIRCUMFERENCES

Note--Where the taper applied to the butt circumferences calculate to a circumference at the tip of less than 16 in. (406 mm), the individual values have been increased to 16 in. (406 mm) to assure a minimum of 5-in. (127 mm) tip for purposes of driving.

Required Minimum Circumference, in. (mm) 3 ft (914 mm) from butt	22 (559)	25 (635)	28 (711)	31 (787)	35 (889)	38 (965)	41 (1041)	44 (1118)	47 (1194)	50 (1270)	57 (1448)
Length, ft (m)				Minimum Tip Circumference, in. (mm)							
20(6.1)	16.0 (406)	16.0 (406)	16.0 (406)	18.0 (457)	22.0 (559)	25.0 (635)	28.0 (711)				
30(9.1)	16.0 (406)	16.0 (406)	16.0 (406)	16.0 (406)	19.0 (483)	22.0 (559)	25.0 (635)	28.0 (711)			
40(12.2)				16.0 (406)	17.0 (432)	20.0 (508)	23.0 (584)	26.0 (660)	29.0 (737)		
50(15.2)					16.0 (406)	17.0 (432)	19.0 (483)	22.0 (559)	25.0 (635)	28.0 (711)	
60(18.3)						16.0 (406)	16.0 (406)	18.6 (472)	21.6 (549)	24.6 (625)	31.6 (803)
70(21.3)						16.0 (406)	16.0 (406)	16.0 (406)	16.2 (411)	19.2 (488)	26.2 (665)
80(24.4)							16.0 (406)	16.0 (406)	16.0 (406)	16.0 (406)	21.8 (554)
90(27.4)							16.0 (406)	16.0 (406)	16.0 (406)	16.0 (406)	19.5 (495)
100(30.5)							16.0 (406)	16.0 (406)	16.0 (406)	16.0 (406)	18.0 (457)
110(33.5)										16.0 (406)	16.0 (406)
120(36.6)											16.0 (406)

TABLE 20 - ASTM D25-79 CIRCUMFERENCES AND DIAMETERS OF TIMBER PILES (continued)

20b: SPECIFIED TIP CIRCUMFERENCES WITH MINIMUM BUTT CIRCUMFERENCES

Tip Circumference in. (mm) Required Minimum Length, ft (m)	16 (406)	19 (483)	22 (559)	25 (635)	28 (711)	31 (787)	35 (889)	38 (965)
	Minimum Circumference 3 ft. from Butt, in. (mm)							
20(6.1)	22.0 (559)	24.0 (610)	27.0 (686)	30.0 (762)	33.0 (838)	36.0 (914)	40.0 (1016)	43.0 (1092)
30(9.1)	23.5 (597)	26.5 (673)	29.5 (749)	32.5 (826)	35.5 (902)	38.5 (978)	42.5 (1080)	45.5 (1156)
40(12.2)	26.0 (660)	29.0 (737)	32.0 (813)	35.0 (889)	38.0 (965)	41.0 (1041)	45.0 (1143)	48.0 (1219)
50(15.2)	28.5 (724)	31.5 (800)	34.5 (876)	37.5 (953)	40.5 (1029)	43.5 (1105)	47.5 (1206)	50.5 (1283)
60(18.3)	31.0 (787)	34.0 (864)	37.0 (940)	40.0 (1016)	43.0 (1092)	46.0 (1168)	50.0 (1270)	53.0 (1346)
70(21.3)	33.5 (851)	36.5 (927)	39.5 (1003)	42.5 (1080)	45.5 (1156)	48.5 (1232)	52.5 (1334)	55.5 (1410)
80(24.4)	36.0 (914)	39.0 (991)	42.0 (1067)	45.0 (1143)	48.0 (1219)	51.0 (1295)	55.0 (1397)	58.0 (1473)
90(27.4)	38.6 (980)	41.6 (1057)	44.6 (1133)	47.6 (1209)	50.6 (1285)	53.6 (1361)	57.6 (1463)	60.5 (1537)
100(30.5)	41.0 (1041)	44.0 (1118)	47.0 (1194)	50.0 (1270)	53.0 (1346)	56.0 (1422)	60.0 (1524)	
110(33.5)	43.6 (1107)	46.6 (1184)	49.6 (1260)	52.6 (1336)	55.6 (1412)	61.0 (1549)		
120(36.6)	46.0 (1168)	49.0 (1245)	52.0 (1321)	55.0 (1397)	58.0 (1473)			

TABLE 21 - AASHTO M-168 CIRCUMFERENCES AND DIAMETERS OF TIMBER PILES(1)

Length	3 feet from butt: Minimum		3 feet from butt: Maximum		At tip, minimum	
	Circum-ference	Diameter (approx-imate	Circum-ference	Diameter (approx-imate)	Circum-ference	Diameter (approx-imate)
Feet	Inches	Inches	Inches	Inches	Inches	Inches
Douglas Fir, Hemlock, Larch, Spruce or Tamarack						
Under 40 . . .	38	12	63	20	25	8
40 to 50 incl .	38	12	63	20	22	7
51 to 70 incl .	41	13	63	20	22	7
71 to 90 incl .	41	13	63	20	19	6
Over 90[2]. . .	--	--	--	--	--	-
Oak and Other Hardwoods, Cypress						
Under 30 . . .	38	12	57	18	25	8
30 to 40 incl .	41	13	63	20	22	7
Over 40	41	13	63	20	19	6
Cedar						
Under 30 . . .	38	12	69	22	25	8
30 to 40 incl .	41	13	69	22	25	8
Over 40	41	13	69	22	22	7

(1) 1 ft. = 305 mm; 1 in. = 25.4 mm.

(2) Dimensions of piles over 90 feet in length shall be as given in the special provisions.

class B piles (Table 18) indicates that **they** are identical. In Table 22 the limitations placed on the **major** strength influencing imperfections by M-168 are summarized. The imperfection 1 imitations in Table 22 are identical to the **D25-58** class B piles.

The preservative treatment requirements for timber piles are frequently specified by referencing a standard material specification. **AASHTO** has an Interim Specification for Preservatives and Pressure Treatment Process for Timber (AASHTO Designation M-133-73) which references the American Wood Preservers Association (AWPA) standards for both preservative quality and preservative treatment processes and results. Hence, treatment in accordance with AWPA standards is assumed herein to the extent that they influence evaluation of AASHTO piling stresses.

Influence of Load Duration, β - The strength of wood is strongly dependent on the duration that the applied load must be **sustained.** The approximate relationship shown in Figure 24 is universally used in wood, design to account for the influence of load duration (Wood Handbook, 1974; Gurfinkel, 1981; Sunley, 1974; Boyd, 1962; **American** Institute of Timber Construction, 1974; National Forest Products Association, 1977; American **Railway** Engineering Association, 1979; **AASHTO** 1977). While this relationship was developed from bending tests on dry small clear specimens, limited **data** indicate it is equally applicable to other properties (Wood, 1951; et. al., 1957; Sugiyama, 1967; and the Wood Handbook, 1974). There are some indications that the influence of load duration may be greater at higher moisture contents, the prevailing condition for timber piles (Sugiyama, 1967 and the Wood Handbook, 1974) , However, it is not possible to quantify the effect at this time. It is apparent that further research on load duration is needed, particularly for compression parallel to grain in the green condition. Timber piles are frequently heated **to** high temperatures during the treatment conditioning process, with a resultant permanent strength loss as discussed in the following section. Research on the load duration behavior of wood permanently weakened by the conditioning process is needed because in current wood design procedures the influence of these two variables is taken as the product of the two ind ividua 1 adjustments , and can account for a 55 percent strength reduction in some species.

Under very short load durations such as the blow of a pile hammer, the value of the load duration adjustment factor, β , is greater than 1.0. Additional information on the strength of wood under very short stress durations has been reported by Wilkinson (1968) and Keeton (1968). Wilkinson reports impact tests on small clear specimens at rise-times to failure of approximately 0.25 millseconds which resulted in β -values ranging from 1.9 to 3.2, with an average value of 2.5. Keeton reports test data on Douglas Fir small clear specimens indicating 8 -values of 1.3 to 1.5 for loading rates compatible with stress wave rise-times experienced during pile driving.

At design stress levels, fatigue failure due to repeated loading is not generally a problem in wood (Horner et. al., 1957). However, peak pile stresses during driving are considerably higher than static design stress levels, often approaching or exceeding the ultimate strength of the material. At these high driving stresses, the potential for fatigue failure increases. In addition, imperfections such as knots, checks, shakes, and splits have a more adverse effect on the fatigue and dynamic

TABLE 22 - LIMITATION OF MAJOR STRENGTH INFLUENCING IMPERFECTIONS FOR AASHTO M-168

Imperfection	Limitations
Knots	(a) For Piles of 50-ft (15.2 m) length or less and for **3/4** is of length below the butt of piles longer than 50 ft. Largest knot diameter, k_L, not greater than 4 inches (102 mm.) sum of knot diameters in 1 ft. (305 mm.) k_S, not greater than 8 inches (203 mm.). Largest knot ratio, k_L/D, not greater than **1/3** Sum of knots ratio, k_S/D, not greater than **2/3** Cluster knots not permitted. **(b)** For lower **1/4** of length of piles longer than 50 ft. Largest knot diameter, k_L, not greater than **5** inches (127 mm.) Sum of knot diameters in 1 ft., k_S, not greater than 10 inches (254 mm.). Largest knot ratio, k_L/D, not greater than **1/2** sum of knots ratio, k_S/D, not greater than 1 Cluster knots not permitted.
Splits	Shall not be longer than the butt diameter.
Shakes	Shakes in outer half of radius of the butt, measured along the annual ring shall not **exceed 1/3** of the butt circumference.
Spiral Grain	Twist of grain in any 20 ft (6.1 m.) length shall not exceed one-half the pile circumference at the mid-point of the measured length.

strength of wood than under static loading conditions (Wood Handbook, 1974 and Horner et. al., 1957).

Because a pile may experience several hundred blows during installation, the use of extremely high strength increases under impact, such as reported by Wilkinson (1968), for the evaluation of allowable driving stresses needs to be tempered by the potential for: (1) low cycle fatigue, and (2) the cumulative effect of the 'load duration phenomenon (Wood Handbook, 1974). Additional research on the behavior of wood under repeated high stress impact such as experienced during pile driving is needed. In order to be meaningful to the designer, such research should include tests on members containing imperfections as encountered in practice as well as clear wood specimens.

For deriving allowable design stresses in timber piles, it is recommended that the β -values given in Figure 24 (and listed in Table 17) be used. This results in a $-value of 0.625 for normal load duration and 0.5625 for long-term loading conditions. During design and installation, piles are frequently subjected to load tests for one to two days duration. For evaluation of timber pile strength during testing a 8 -value of 0.83 is appropriate.

Frequently, during design stages when pile driveability is being considered, or during construction when pile damage is being experienced, or pile load tests fail, the engineer faces the problem of evaluating the maximum allowable driving stress. The selection of an appropriate β -value for driving stresses is difficult because of the lack of data on repeated impact, as experienced during pile driving , and the absence of impact tests on specimens containing imperfections. It would appear that for evaluation of driving stresses a β -value of 1.2 can be used for most normal driving conditions. In ideal conditions a 8 -value of up to 1.5 might be attained.

Influence of High Temperatures on Strength of Timber Piles, ψ - Temperatures in excess of approximately 150°F (68°C) can result in a permanent reduction in wood strength, and temperatures between 700F (210C) and approximately 1500F (680C) can result in an immediate effect, as indicated previously in the discussion of factors influencing clear wood strength. The effect under high moisture contents, the case in most pile environments, is greater than for dried wood. In applications such as industrial manufacturing plants where high temperatures are prevalent (for example coke ovens or steel blast furnace foundations) the influence of high temperatures over long durations can be sufficient cause to reject the use of timber piles as foundation support, or at minimum, lead to major allowable stress reductions. Several cases of unsatisfactory foundation behavior have resulted when timber piles have been used in such environments (New England Construction, March 27, 1978 and Fuller, 1978). For the environments of most highway structures the consideration of temperature reductions for timber piles subsequent to installation does not need to be considered. However, permanent strength loss during the treatment process prior to installation can be large, and must be considered, as indicated below.

In many applications timber piles are treated prior to installation to protect them from deterioration under the destructive action of fungi, insects, or marine borers. If treatment is to serve its intended function, the required preservative penetration and retention must be obtained during the treatment process. The process that has been developed to treat timber piles frequently subjects the piles to temperatures on the order of 200°F (93°C) to 250°F (12loC)which can lead to a permanent reduction in pile strength (Wood Handbook, 1974; ASTM D245-74, Brown et. al., 1952; Hunt and Garratt, 1953; Thompson, 1969; and Wilkinson, 1968).

The strength reduction as a result of treatment is dependent on: (1) the process (nature of the heating medium) used, (2) the moisture content and dimensions of the treated pieces, (3) the temperatures used in the processes, (4) the exposure period to the temperature, and (5) the property being considered. The strength reduction is also somewhat dependent on species, with greater reductions occurring in hardwoods than in softwoods (Wood Handbook, 1974). Because the influence of temperature is cumulative, the exposure period should be considered as a summation of all cycle times when the piles are subjected to more than one cycle at elevated temperature during conditioning and treatment. From a design standpoint, the limiting ranges of these variables are defined by the specification under which the piles will be treated, which is almost invariably AWPA C3 (American Wood Preservers Association). Armstrong (1979) reviewed the influence of treatment on the axial compression strength of timber piles under the then current AWPA standards and recommended strength reduction factors, ψ -values, of 0.75 for Southern Pine and 0.9 for Douglas fir and oak piles treated in accordance with the penissable treatment processes, temperatures, and exposure periods specified in AWPA C3.

The major portion of the strength reduction during treatment appears to be related to the conditioning process used (Wilkinson, 1968). Principal conditioning processes currently used with timber piles are air seasoning, the Boulton process (heating in the preservative), steaming, or a combination of the preceding processes. Although piles could be conditioned by kiln drying, it is not presently a general practice.

While there is not a great deal of information available, it appears that piles properly conditioned by air seasoning do not experience a major strength loss during the treatment process (Wood et. al., 1960; Wilkinson, 1968; and Thompson, 1969a). Hence a ψ -value of 1.0 is recommended for air seasoned piles. Information on kiln dried piling is also limited; the information that is available (Wilkinson, 1968 and Thompson 1969b), indicates that the loss may range from zero to 15 percent and varies as a function of drying temperatures and duration. More information on kiln drying of piles is needed to define the influence of drying temperature and temperature duration on pile strength. Unt il such information becomes available, a ψ - value of 0.9 is recommended for both compression and bending in the event kiln drying is used.

Numerous test programs on round timber piles and poles (Eggleston, 1952; Wood et. al. 1960; Wilkinson, 1968; Peterson, undated; Thompson, undated; Thompson, 1969a; Thompson, 1969b; and Walford, 1980) have been conducted that allow the influence of both the Boulton process and steaming to be evaluated analytically. A summary of ψ -values indicated by tests on piles and poles conditioned by the Boulton process are presented in Table 23 The indicated ψ -values are, for practical purposes,' the same in both compression and bending and for small clears and full size sections. The conditioning temperature and duration were not reported for the WWPI (Western Wood Preservers Institute) pile tests (Peterson, undated) which show $-values which are generally in excess of unity. The other data at temperatures of approximately 2000F (930C)and 14 to 27 hour exposure

TABLE 23: Summary of Data on Use of the Boulton Process

$$\psi = \frac{\text{Strength of treated Section}}{\text{Strength of untreated Sections}}$$

Species	Treatment Conditions		Adjustment ratio, ψ				REMARKS
			Bending		Compression		
	Temperature °F	Exposure time Hours	Piles or Poles	Small Clears	Piles or Poles	Small Clears	
Douglas Fir	180 to 203 then 200 to 208	2 hr. then 25.3 hr.	0.94	0.90	---	0.93	30' Poles Wood et al, 1960 ASTM Pole Study
			0.94	0.91	---	-0.93	25' Poles
			0.85	0.87	---	0.84	55' Poles
Western Larch'	180 to 190 then 200	4.0 then 18.0	0.83	0.86	i-i	0.85	30' Poles
			0.86	0.90	---	0.92	25' Poles
			0.92	0.90	---	0.86	55' Poles
Douglas Fir	190	14.0	0.93	0.94	1.06	0.98	Wilkinson, 1968, 50 ft Piles
			---	---	0.98	0.97	untreated strength estimated on the basis of ASTM D2555
Douglas Fir	Not Known	Be-	---	---	1.05	---	Peterson, undated, pile tips
			---	---	1.12	---	
			---	---	1.03	---	
			---	---	1.06	---	
			Be-	---	1.02	---	
			---	---	0.92	---	
			---	---	1.05	---	

periods indicate $-values ranging from approximately 1.0 to '0.83. Armstrong, 1979, recommended a ψ -value Of 0.9 for compression in piles treated by the Boulton process, primarily on the basis of pile-section tests by Wilkinson (1968) and the WWPI tests. Considering that current treatment standards (AWPA C3-81) permit conditioning by the Boulton process at 220°F for unlimited duration, the ψ-values from the ASTM Pole Study (200°F [93°C] @ 22 to 27 hrs.) indicates Armstrong's recommendation of 0.9 may be somewhat unconservat ive.

For conditioning at 2200F (104°C) for an unlimited duration'as permitted by specification with the Boulton process, a ψ -value of 0.85 appears more reasonable. Tests at 2200F (104°C) for 24 hrs. to 48 hrs. durations are needed to confirm that this value is adequate. Pending the results of such tests? it appears desirable to be more restrictive on the temperature and duration permitted with the Boulton process, provided that the required treatment penetration and retentions can be obtained. Until more definitive data are developed, a ψ -value of 0.85 is recommended for use in both bending and compression.

To obtain the required penetration and retention in some species, such as southern pine, piles and poles are most frequently conditioned by steaming. A summary of ψ -values from test programs with piles and poles conditioned by steaming is presented in Table 24. The ψ -values from the ASTM Pole Study are approximately the same for both bending and compression of small clear specimens, whereas the values for bending of round poles are approximately 10 to 15 percent less than for the bending of small clear specimen 8. This difference is probably attributable to the increased checking that develops in round pole sections during treatment. The tests by Wilkinson (1968) and Thompson (undated and 1969a) indicate that the ψ -values in compression are approximately the same for small clear specimens and pile sections. This is reasonable because shakes and checks have only a minor influence on compression parallel to grain.

Based on the unfavorable results in the ASTM Pole Study (Wood et. al., 1960), AWPA lowered the maximum steaming temperature permitted from 259°F (126oc) to 2450F (1180C). In 1979 they lowered the permitted maximum duration from 22 hours to 15 hours. Data in Table 24 indicate that a reasonable ψ -value for 2450F (1180C) and 15 hours duration is approximately 0.75, if the Wilkinson (1968) tests are discounted. It is recommended that a ψ -value of 8.75 be used in both compression and bending. The larger strength reduction observed in bending of poles was caused by the increased imperfections resulting from steaming. This will be accounted for separately by the strength ratio, ϕ_b.

In the preceding paragraphs ψ -values were recommended for use with the various conditioning processes permitted in, AWPA C3-81; as follows:

Conditioning Process	ψ-Value
Air seasoning	1.0
Kiln Drying	0.9
Boulton Process	0.85
Steaming	0.75

In AWPA C3-81 more than one conditioning process is permitted for the various species. From the designer's standpoint, when treatment of piles

TABLE 24 - SUMMARY OF DATA ON STEAMED POLES AND PILES

ψ = Steamed Strength / Unsteamed Strength

Species	Treatment Condition: Temperature °F	Exposure Time Hours	Adjustment Ratio. Bending: Piles or Poles	Bending: Small clears	Compression: Piles or Poles	Compression: Small clears	Remarks
Longleaf			0.75	0.89	--	0.79	30' Poles Wood et.al. 1960
Slash	259	10.0	0.82	0.92	--	0.89	30' Poles ASTM Pole Study
Longleaf			0.75	0.88	--	0.74	25' Poles
Shortleaf	259		0.64	0.81	--	0.79	30' Poles
Loblolly		13.5	0.62	0.79	--	0.81	30' Poles
Shortleaf			0.63	0.84	--	0.82	25' Poles
Lonnleaf	259		0.75	0.80	--	0.96	55' Poles
Shortleaf		15.0	0.80	0.84	--	0.86	55' Poles
	212		0.82	0.94	--	0.99	25' Poles
				(0.93)		(0.88)	
	240		0.87	0.92	--	0.88	25' Poles
				(0.82)		(0.81)	
	242	6.0	0.79	0.95	--	0.94	25' Poles
				(0.80)		(0.79)	
	259		0.77	0.89	--	0.89	
				(0.72)		(0.71)	
	260		0.72	0.83	--	0.86	numbers in parenthesis are
				(0.75)		(0.71)	from 3x3 in. sticks
Longleaf	271		0.72	0.85	--	0.84	treated with poles
				(0.73)		(0.74)	
	224	15.0	0.78	0.91	--	0.91	
				(0.83)		(0.80)	

continued

TABLE 24 - SUMMARY OF DATA ON STEAMED POLES AND PILES

$$\psi = \frac{\text{Steamed Strennth}}{\text{Unsteamed Strength}}$$

Species	Treatment Condition: Temperature °F	Treatment Condition: Exposure Time Hours	Adjustment Ratio. Bending: Piles or Poles	Adjustment Ratio. Bending: Small clears	Adjustment Ratio. Compression: Piles or Poles	Adjustment Ratio. Compression: Small clears	Remarks
Longleaf	243	15.0	0.79	0.85 (0.79)	--	0.87 (0.75)	Wood et al. 1960 ASTM Pole Study (continued) 25' poles
	244		0.68	0.77 (0.73)	--	0.73 (0.71)	
	259		0.66	0.70 (0.76)	--	0.76 (0.78)	
	269		0.59	0.69 (0.70)	--	0.65 (0.65)	
	274		0.61	0.70 (0.62)	--	0.76 (0.61)	
Southern Pine	245	15.0	--	--	0.67	0.65	Wilkinson, 1968, 50ft piles untreated-strength estimated on the basis of ASTM D2555
			0.70	0.67	0.85	0.79	
Southern Pine	245	15.8	--	--	0.77	0.80	Thompson, undated, and Thompson, 1969b, 50ft piles *partially seasoned prior to steaming
	240	8*	--	--	0.82	0.86	
Southern Pine	245	14	0.72	0.79	0.82	0.80	Thompson, 1969b, 30ft poles ratio steamed to kiln dried
Corsican Pine	239	4.7	0.75	0.95	--	--	Walford, 1980 Summary of available data in Hew Zealand
		12.7	0.79	0.77	--	--	
	250	7.4	0.83	0.82	--	--	
		7.3	--	0.85	--	--	
		7.6	0.81	0.85	--	--	
	261	6.5	0.85	0.86	--	--	
		12.2	0.68	0.71	--	--	

continued

TABLE 24 - SUMMARY OF DATA ON STEAMED POLES AND PILES

$$\psi = \frac{\text{Steamed Strength}}{\text{Unsteamed Strength}}$$

Species	Treatment Condition		Adjustment Ratio,				Remarks
			Bending		Compression.		
	Temperature °F	Exposure Time Hours	Piles or Poles	Small clears	Piles or Poles	Small clears	
Radiata Pine	302		--	0.74	--	--	Walford, 1980 (continued) Summary of Available Data in New Zealand
	320		--	0.54	--	--	
	338	1.25	--	0.49	--	--	
	356		--	0.45	--	--	
Corsican Pine	246	7.5	0.73	0.89	--	--	
	241	6.0	0.88	--	--	--	
	250	8.0		0.81	--	--	
Radiata Pine	259	1.0	--	0.83	--	--	
		4.0	--	0.72	--	--	

is specified, the attainment of the required penetration and retention should be the primary objective because proper treatment is essential to Preserve the integrity of the pile. Under the provisions of AWPA C3 the supplier is given the freedom, within limits, of selecting the most appropriate conditioning process to meet the required performance requirements of penetration and retention. Hence, unless the designer is willing to assume the responsibility of specifying the conditioning process, and is willing to accept the resulting penetration and retention, he must assume the conditioning process associated with the lowest allowable stress during design. If the treatment retention and penetration are specified, and the supplier is allowed the freedom to select the conditioning process (within limits of AWPA C3), the design ψ -value also becomes species-dependent and not just a function of process. Recommended ψ -values for various species are summarized in Table 25 based on the conditioning extremes currently permitted in AWPA C3-81.

Influence of Imperfections on Timber Pile Strength, ϕ - Because imperfections such as spiral and sloping grain, knots, checks, shakes and splits are natural growth characteristics of trees, they are present to, some degree in all timber piles. While most references on timber piles and poles indicate the influence of imperfections on round timber members is less than for sawn lumber, few give.quantitative recommendations for evaluating this reduction. The strength reducing effects of imperfections is not as great in round timber members because the sloping fibers are not generally cut at the pile surface as in sawn lumber, and knot holes resulting from fallen knots are less likely to develop. The use of the ASTM D245 strength reductions with round timbers may be over conservative, but use of smaller reductions can be rationally justified only if verified by actual test data. In the following paragraphs results of the various test programs on round timber piles and poles are reviewed and recommendations for strength reductions applicable to round timber piles are made to the extent available data permit.

Several test programs on round timber piles and poles (Wood et. al., 1960; Wilkinson, 1968; Thompson, undated; Thompson, 1969a; Thompson, 1969b; and Walf ord, 1980) have been conducted which included tests on poles or piles as well as tests on small clear wood specimens. In most cases only the average strengths are reported; in a few the individual strengths for each specimen are reported, Only Wilkinson, Thompson, and Wa lf ord (Wilkinson, 1968; Thompson, undated; and Walford, 1980), provided details, on the imperfect ions present.

For compression tests the ratio of the failure stress in the pile or pole to the failure stress of the companion small clear specimen is a direct measure of the strength ratio, ϕ_c, associated with the imperfections present. However, for bending tests, the ratio of the pile or pole failure stress to the companion small clear specimen stress represents the combined influence of the strength ratio, ϕ_b, associated with imperfections and the adjustment factor for influence of size and shape on the modulus of rupture, f . Hence, the strength ratio associated with imperfections in bending, ϕ_b, can be evaluated from test data only if the influence of size and shape is known.

In sawn lumber the bending strength, as indicated by the modulus of rupture, decreases as the depth of the member increases (Bohannan, 1966). Although there is a depth influence in round timbers, there is no evidence

TABLE 25 - **RECOMMENDED STRENGTH REDUCTION FACTORS** FOR THE INFLUENCE OF **CONDITIONING, ψ** [1]

SPECIES[2]	CRITICAL CONDITIONING PROCESS PERMITTED	RECOMMENDED STRENGTH REDUCTION FACTOR, ψ
Southern Pine	Steaming at 2450F for 15 hours	0.75
Pacific Coast Douglas Fir		
Red Pine		
Ponderosa Pine	Heating in Preservative at 220°F - no time limit	0.85
Lodge-pole Pine		
Western Larch		
Jack Pine		
Interior Douglas Fir		
Oak		

[1] Based on conditioning permitted by AWPA C3

[2] Requirements for other species not covered by AWPA C3

that it is the same as for sawn lumber. The modulus of rupture is also dependent on form or shape of the cross-section. For round members the modulus of rupture is approximately 1.18 times the modulus of rupture for square members (Wood Handbook, 1974; Gurfinkel 1981; and National Forest Products Association, 1977). If the adjustment factor for sawn lumber given by Equation 17 and the form factor of 1.18 are used, the combined adjustment factor for size and form for round members with diameters in the range of 8 to 12 inches (200 to 300mm) varies from 1.01 to 0.97 or essentially unity. Because the majority of the poles and piles in the various test programs reviewed have diameters in the 8 ta 12 inch (200 to 300mm) range, it would appear the ratio of pole or pile bending strength to small clear strength reflects primarily the influence of imperfections because the combined influence of size and form is approximately unity. In the following section the ratio of pile or pole bending strength to small clear specimen strength will be denoted by the symbol, ϕ_b, even though there may be some minor influence of size and form.

During the 1950's an extensive research program on the strength of wood poles was conducted under the sponsorship of ASTN. This research program indluded teats on southern pine, Douglas fir, western larch, lodgepole pine, and western red cedar species, and covered teats on 620 full-sized poles and approximately 14,000 tests on small clear specimens. Results of this teat program (referred to hereafter as the ASTM Pole Study) were summarized and analyzed in a final report by Wood, Erickson, and Dohr (1960).

All poles in the ASTM Pole Study were machine shaved. Small clear specimens were obtained near the butt end of the poles, or approximately 8 feet below the location referred to as the groundline in the test setup. The poles were tested by both machine testing and cantilever methods. While the majority of the teats were performed on 30-ft (9.1 m) poles, limited testing was also conducted on poles of 25, 45, and 55 ft. (7.5, 13.7, and 16.8 m.) lengths. The distances from the pole butts to the ground line were 5 ft (1.5 m)for 25 ft poles, 5.5 ft (1.7 m) for 30 ft poles, 6.5 ft (2.0 m) for 45 ft poles (used only on lodgepole pine), and 7.5 ft (2.3 m) for the 55 ft poles,

The modulus of rupture at the ground line for both the poles and the small clear specimens are reported. The ratio of these two strengths are an estimate of the strength ratio, ϕ_b. The values of the strength ratio, based on average pole strength and average small clear strength, indicated by the ASTN Pole Study, are summarized in Table 26. The data in Table 26 indicates ϕ_b-values ranging from 0.67 to 0.99; there are some indications that ϕ_b may decrease with treatment and increasing pole length. However, the reliability of using the ASTN Pole Study data to evaluate the influence of imperfections is highly questionable.

The pole modulus of rupture reported in the ASTM Pole Study is a value calculated at the ground line for the applied failure load. While tbe maximum moment occurs at the ground line, the bending resistance of the pole generally decreases from the pole butt to the tip because of: (1) the decrease in section modulus toward the tip due to pole taper, (2) the decrease in small clear strength toward the tip, and (3) the general increase in knot size and frequency toward the tip. Therefore, the location of pole failure, the section with lowest strength to applied moment ratio, should occur at some distance above the ground line. This

TABLE 26: BENDING STRENGTH RATIOS FROM ASTM POLE STUDY (WOOD et. al. 1960)[1]

SPECIES	Strength Ratio, ϕ_b [2] 25 FT. POLES (7.6M)	30 FT. POLES (9.1M)	55 FT. POLES (16.8M)
	UNTREATED POLES AND SMALL CLEARS		
Longleaf and slash	0.96	0.96	0.95
Shortleaf and loblolly	0.99	0.95	0.89
Douglas fir	0.95	0.92	0.98
Western larch	0.89	0.90	0.85
Lodgepole pine	0.86	0.89	0.83[3]
Western redcedar	0.88	0.91	0.67
	TREATED POLES AND SMALL CLEARS		
Longleaf and slash	0.86	0.86	0.86
Shortleaf and loblolly	0.87	0.79	0.94
Douglas fir	0.89	0.82	0.87
Western larch	0.86	0.92	0.91
Lodgepole pine	0.83	0.87	0.85[3]
Western redcedar	0.77	0.82	0.72

[1] Poles of extremely low and high specific gravity excluded
[2] Ratio of average modulus of rupture at ground line to average modulus rupture of small clear specimens
[3] Lodgepole pine piles were 45 ft. rather than 55 ft.

behavior was observed in the ASTM Pole Study where the actual pole failure locations ranged from zero to 12 ft (3.66m) above the ground line with the majority occurring at approximately 1 to 3 ft (0.3 to 0.9m) for the 25 and 30 ft (7.6 and 9.1m) poles. For the longer poles, failure occurred at 1 to 20 ft (0.9 to 6.1m) above the ground line, with the majority being on the order of 9 to 14 ft(2.7 to 4.3m). Because actual failure was not reached at the ground line in many cases, the real average failure stresses at the ground line would be higher than the reported ground line stresses, resulting in ϕ_b-values at the ground line slightly higher than those given in Table 26. Similarly, failure stresses at the actual failure locations would likely be slightly less than the reported ground line stresses due to the reduced moment arm, resulting in ϕ_b-values at the actual failure section being slightly less than those in Table 26.

The data presented in the ASTM Pole Study Final Report (Wood, et. al., 1960) is insufficient to permit calculation of the failure stresses at the actual failure locations, because the actual failure locations and diameters at the failure locations are not given. Bohannan (1971) re-analyzed the original test data for the 55-ft (16.8 m) poles, calculating the bending strength at the failure location for all poles tested by the cantilever method, and for the western red cedar poles tested by the machine method. The results of Bohannan's analysis are summarized in Figure 30, where the actual failure stress is expressed as a percentage of the reported ground line stress and the failure location is given as a percentage of height above the ground line. The data in Figure 30 shows that the actual failure location was generally above the ground line, and that the actual failure stress was generally less than the reported ground line stress for the 55-ft poles. The data also indicate a general trend of decreasing strength with increasing height above the ground line, resulting from the decreasing clear wood strength and increasing frequency of knots. The 25 and 30-ft (7.6 and 9.1m) poles had a greater tendency to break near the ground line; hence, the difference between the reported ground line stress and the actual failure stress for the shorter poles was probably less than indicated by Figure 30.

Detailed knot dimensions are not presented in the ASTM Pole Study; therefore, even if it were possible to refine the strength ratio by re-evaluating the failure stress, quantitative correlations with the extent of the defects present would not be possible. However, the poles with large knots were studied during the ASTM Pole Study in an attempt to formulate a general rule for estimating the effect of knots on strength (Wood, et. al., 1969). The approximate rules for evaluating the strength-reducing effect of the largest knot (k_L) and sum of knots (k_S) recommended by ASTM Pole Study are given in equations 23 and 24:

$$\phi b = 1 - 0.64\ k_L/D \qquad \text{EQ. 23}$$

$$\phi b = 1 - 0.32\ k_S/D \qquad \text{Eq. 24}$$

The ASTM Pole Study provided no approximations for other imperfections such as checks, shakes or slope of grain.

Wilkinson (1968) reported the results of a test program on treated 50-ft (15.2m) Douglas fir, southern pine, and red oak piles. Fifteen piles of each species were tested to obtain compression and bending data on both full cross-section and small clear specimens. Sections 5 ft (1.5m) in

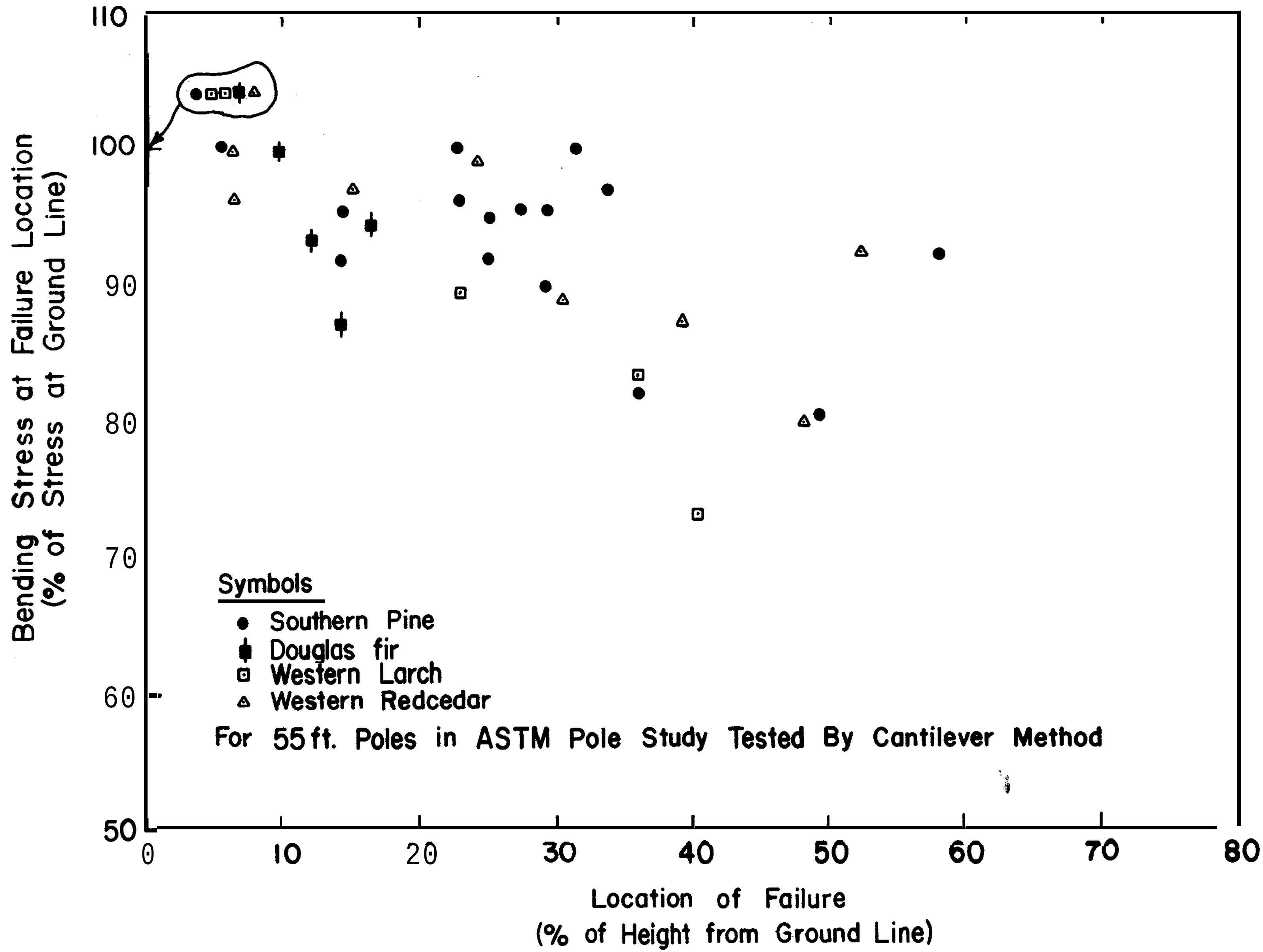


Figure 30 : Stress at Failure vs. Location of Failure
(After Bohannon 1971)

length were removed from the tips and butts of the piles to provide 3-ft (0.9m) long full cross-section compression specimens and small clear wood specimens. The remaining 40-ft (12.2 m) pile section was tested in bending. Small clear bending specimens were obtained as close to the failure location as possible, after the bending test on each pile.

The 40-ft (12.2 m) bending specimens were tested over a 36-ft (11m) span by loading at the quarter points. The loads were proportioned to the section modulus at each load point so that an approximately equal bending stress developed between the two loading points. Bending failure usually occurred near the tip load point because of the larger concentration of knots toward the tip. The bending failures "started with compression wrinkles near knots, followed by splintering tension failures," (Wilkinson, 1968). The modulus of rupture of each individual pile was reported by Wilkinson, but only the average small clear bending strength was reported for each of the species, Hence, it is only possible to calculate the bending strength ratio based on the average bending stresses. Bending strength ratios from Wilkinson's tests are presented in Table 27 for the three species. Knot dimensions near the bending failures were not reported.

TABLE 27: BENDING STRENGTH RATIOS REPORTED BY WILKINSON (1968)[1]

SPECIES	STRENGTH RATIO, ϕ_b [2]
Southern Pine	0.90
Douglas-fir	0.88
Red Oak	0.82

[1] From bending tests on treated piles
[2] Ratio of average modulus of rupture of piles to modulus of rupture of small clear specimens

The compression failure stress for the 3-ft (0.9m) pile tip and butt sections was calculated on the basis of the smallest end of each specimen, and the results of each specimen were reported. Because only the average small clear strengths were reported, it is only possible to calculate strength ratios based on average stresses for each species. Wilkinson also reported the average largest knot diameter, and the average largest summation of knot diameters in 1 foot (305 mm) of length for the butt and tip sections of each species. Compression strength ratios, and the corresponding knot ratios, are summarized in Table 28 for the butt and tip specimens reported by Wilkinson (1968). The strength ratios presented in Table 28 include corrections to the strength data contained in the original report (Wolfe, 1978).

Wilkinson (1968) also reported results of compression tests on 3-ft (0.9m) pile sections and companion small clears performed to investigate the influence of kiln drying. The test specimens were obtained by cutting fifteen 30-ft (9.1m) slash and long-leaf piles into 5 ft (1.5m) sections. One-third of the sections were tested green, one-third were kiln-dried prior to testing, and one-third were kiln-dried and treated prior to

TABLE 28 - COMPRESSION STRENGTH RATIOS AND CORRESPONDING KNOT RATIOS REPORTED BY WILKINSON (1968) [1]

Species	Location	Strength Ratio, ϕ_c [2]	Knot Ratios[3] Largest Knot, kL/D[4]	Sum of Knots, kS/D[5]
Southern pine	Butt	0.97	0.00	0.0
Southern pine	Tip	0.93	0.30	0.77
Douglas fir	Butt	0.98	0.04	0.12
Douglas fir	Tip	0.93	0.15	0.53
Red oak	Butt	0.97	0.06	0.13
Red oak	Tip	0.86	0.38	0.90

[1] From compression tests on 50 ft. (15.2 M) treated piles
[2] Ratio of average pile section crushing strength to average small clear crushing strength
[3] Based on average diameter of small end of specimen
[4] Ratio of average largest knot diameter to average pile diameter
[5] Ratio of average sum of knots in 1-ft. (305 mm) to average pile diameter

TABLE 29 - COMPRESSION STRENGTH RATIOS FROM 30 FT. (9.1 M) SLASH AND LONGLEAF PILES (WILKINSON, 1968)[1]

Distance Below Butt, Feet	Strength Ratio, ϕ_c [2] Average	Standard Deviation
2.5	0.96	0.07
7.5	0.96	0.03
12.5	0.95	0.05
17.5	0.94	0.06
22.5	0.93	0.09
27.5	0.92	0.09

[1] From tests on green, kiln-dried, and kiln-dried-treated pile sections
[2] Ratio of average pile section crushing strength to average small clear crushing strength

testing. The failure stress of each pile and small clear specimen was reported along with the strength ratio for each section. Average strength ratios and the standard deviation of the strength ratios at various positions along the pile axes are summarized in Table 29. The trend for the strength ratio to decrease, and the variability to increase, with increasing distance below the pile butt is a reflection of increasing knot size and frequency of knots toward the tip. The higher strength ratios indicated for these tests is to be expected because the piles were selected to be relatively free of knots, and one-third of the specimens had no knots. Detailed knot dimensions for these specimens were not reported.

An extensive program of compression tests on short pile sections and small clear specimens from clean peeled, southern pine, pile tips were performed at Mississippi State University and reported by Thompson (Thomspon, undated; and 1969a). This program consisted of three test series of 50 piles each. The three test series were from different sources and were subjected to different treatment schedules. The piles were selected at random from regular stock and conformed to ASTM Standard D25-58. Two of the test series were from piles of 50 ft (15.2m) lengths and the third series was from 35 ft (10.7m) piles. The piles had a minimum tip diameter of 6 inches (150mm), and a maximum diameter of 10 inches (250mm) at a distance 10 ft (3m) above the tip.

The 5-ft (1.5m) tip section was cut from each pile prior to treatment and a second 5-ft (1.5m) test section was cut from the tip end after treatment. Pile test specimens 3 ft (0.9m) in length were obtained from each 5 ft (1.5m) section and small clear wood specimens were obtained from one-half of the 5 ft (1.5m) sections. In total, 300 pile sections and 150 small clear specimens were tested during the Mississippi, State University program. In addition to the strength data, the 1 arges t knot diameter and sum of knots in 1 ft (0.3m) were also reported for each 3-ft (0.9m) test section. Average values for the strength ratio and the knot ratios are summarized in Table 30 for the untreated and treated specimens of each test series.

Thompson (1969b) reported results of tests on 30 ft (9.1m) long southern pines poles. All of the poles were machine peeled and selected to be free of knots larger than 1 inch (25mm) in diameter 'within 10 ft (3m) of the butt. A total of 150 poles were selected and divided into 3 groups which were subjected to different conditioning; one of the groups was steam conditioned while the other two were kiln dried. Bending tests were performed on each of the poles. Small clear bending, small clear compression, and full section compression specimens were obtained from one-half of the poles after the bending tests. The 3-ft (0.9m) full-section compression specimens were obtained from a location along the length of the pole so that the diameter of the small end was approximately 7.5 inches (191 mm.); small clear specimens were obtained near the pole butt. Only average strengths were reported, and detailed knot dimensions were not presented.

Strength ratios based on the reported (Thomspon, 1969b) average pole and small clear strengths are summarized in Table 31. While detailed knot data was not presented, the knot dimensions and butt diameter criteria upon which the poles were selected indicates that the largest knot ratio, in the area of bending failure was 0.13 or less. Because the 'small clear compression specimens were obtained from the lower 7 ft (2.1m) near the

TABLE 30 - COMPRESSION STRENGTH RATIOS AND CORRESPONDING KNOT RATIOS (FROM THOMPSON, UNDATED AND 1969a)[1]

Test Series and Condition	Strength Ratio, ϕ_c [2]	Knot Ratios: Largest Knot, k_L/D [3]	Knot Ratios: Sum of Knots, k_S/D [4]
AD&AE - treated	0.88	0.26	0.54
AD&AE - untreated	0.93	0.30	0.59
BA - treated	0.91	0.20	0.40
BA - untreated	0.94	0.24	0.58
CA - treated	0.85	0.23	0.45
CA - untreated	0.85	0.28	0.60

[1] Southern pine poles, series - BA are 35 ft. (10.7 m.) piles. Others are 50 ft. (15.2 m.)

[2] Average ratio of pile section crushing strength to small clear crushing strength

[3] Average ratio of largest knot diameter to pile diameter

[4] Average ratio of sum of knots in l-foot (305 mm.) to pile diameter

TABLE 31 - STRENGTH RATIOS ON 30 FT. SOUTHERN PINE POLES (AFTER THOMPSON 1969b)

Conditioning	Compressive Strength Ratio ϕ_c [1]	Bending Strength Ratio, ϕ_b
Steam Conditioned	0.80	0.77
Kiln dried at 1520F	0.77	0.85
Kiln dried at 182oF	0.78	0.90

[1] Ratio of average pole strength to average small clear strength

pole butts and the 3 ft (0.9m) pole compression specimens were obtained from various locations along the poles, the compression strength ratios in Table 31 may be influenced by strength variation along the poles, as well as the influence of imperfections

Walford (1980) reported results of bending tests on Corsican pine pole segments and companion small clear specimens. The pole segments, obtained from poles at least 10 meters (32.8 ft) long, were 2.5 meters (8.2 ft) in length. Some of the segments were hand peeled while others were machine shaved. The pole segments were subjected to various steam conditioning cycles, and then loaded to failure in four-point bending over a span of 2.25 meters (7.4 ft). During testing the segments were oriented so that the worst knot was on the tension face. After the segment bending tests, the knot and pole dimensions involved in the fracture were recorded and small clear bending specimens were obtained. Bending strength and knot ratios for the Corsican pine pole segments are summarized in Table 32,

Available data from test programs which incorporated both full section specimens as well as small clear wood specimens were reviewed in the preceding paragraphs. The strength ratios indicated by these test programs are summarized in Tables 26 through 32. With the exception of a few programs, it is only possible to calculate strength ratios based on full-section strength and average small clear strength. Because of the primary purpose of most of these programs was to study the influence of treatment on strength, the knot dimensions in the failure area were often either not measured or not presented in the available reports. Hence, only limited information is available to evaluate the scale-effect of knots on round timber strength.

Only the test data presented by Wilkinson (1968) and Thompson (undated, 1969a) provide sufficient data to permit both the compression strength ratio and the corresponding knot ratios to be calculated. The relationships between the average strength ratios and the average knot ratios from these two test programs (Table 28 and Table 30) are indicated by the data points in Figures 31 and 32. Based on these data points, Armstrong (1979) recommended that the strength ratio for piles based on the expressions in Equations 25 and 26 using the maximum permitted knot ratios. These expressions are shown on Figures 31 and 32.

$$\phi_c = 1 - 0.45\ k_L/D \qquad \text{EQ.25}$$
$$\phi_c = 1 - 0.20\ k_S/D \qquad \text{EQ.26}$$

Based on Equation 25, the strength loss in a round timber pile is only approximately 45 percent of the loss indicated by Equation 8 for sawn lumber. Burpee (1958) has indicated that: "knots have only one-half the effect on the strength of natural round sections that they have on the strength of sawn members.** The background for this conclusion was not presented by Burpee.

While the information available on the strength-reducing effects of knots in compression parallel to grain is limited, the available data (Figures 31 and 32) and Burpee's observation indicate that the strength of a timber pile decreases as the size and number of knots increases, but the influence is not as great as in sawn lumber. Until further information becomes available on the strength reduction caused by knots on timber piles, the approximate relationships given in Equations 25 and 26 should be

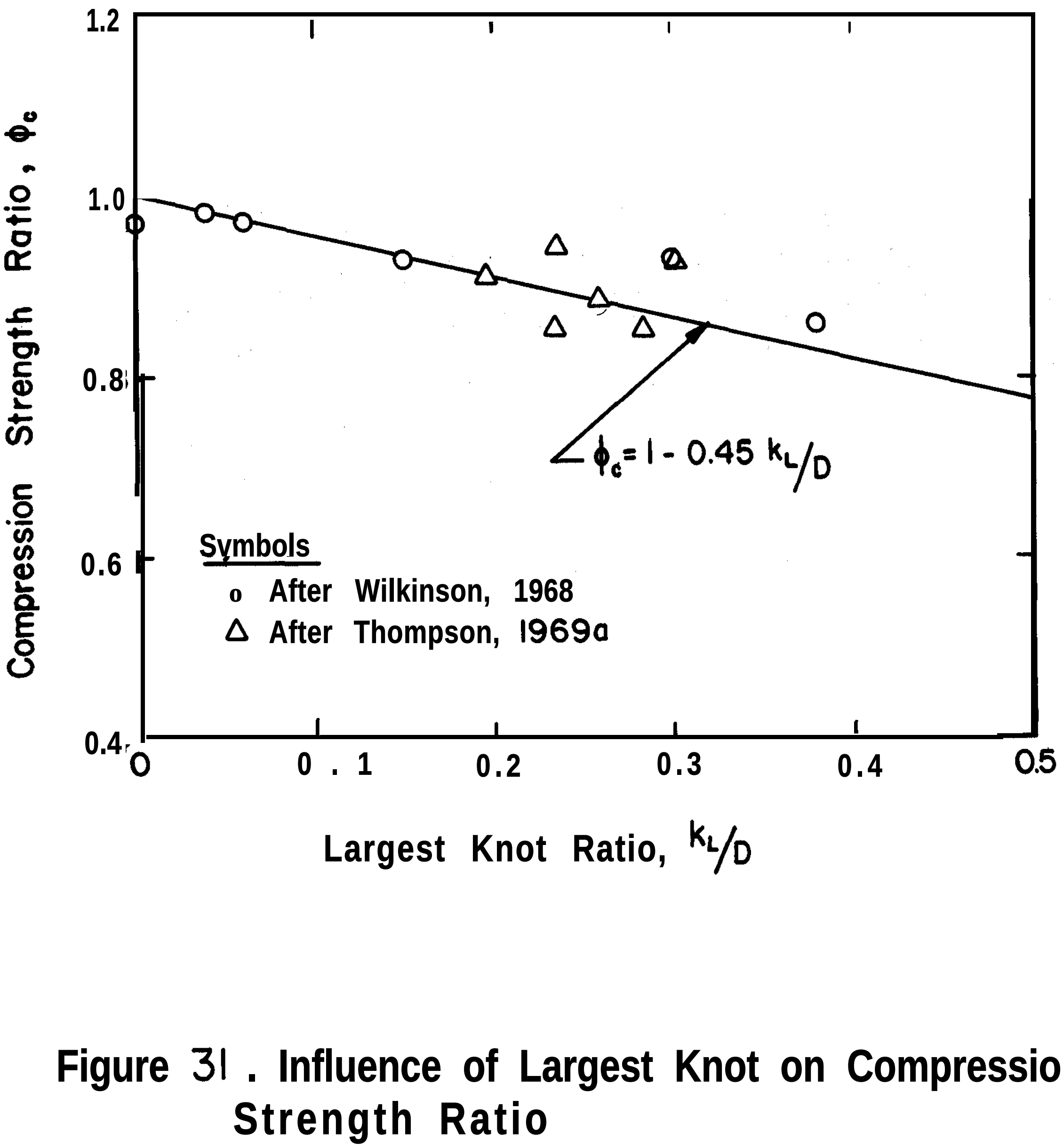
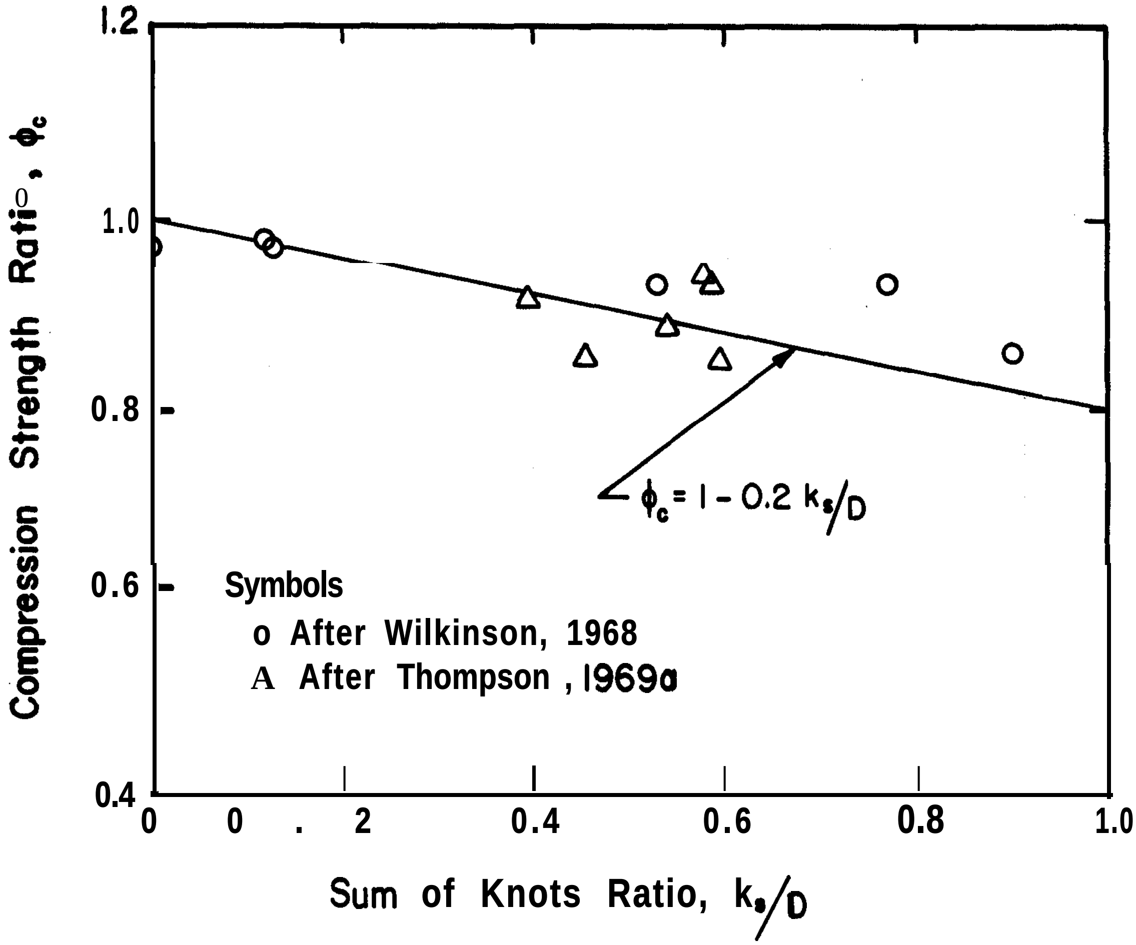


Figure 31. Influence of Largest Knot on Compression Strength Ratio



-Figure 32. Influence of Sum of Knots on Compression Strength Ratio used to estimate the compression strength ratio associated with knots.

The data that can be used to quantify the influence of knots on bending strength is even more limited than that for compression. Only Walford (1980) has reported knot dimensions at the failure location so that the knot ratios corresponding to the strength ratios can be calculated directly. In Figures 33 and 34, the average bended strength ratios and knot ratios from Walford's tests (Table 32) are indicated by the circular symbols. Wilkinson (1968) reported only the knot dimensions of the compression tip and the butt specimens. Assuming the knot ratios of the bending specimens in Wilkinson's tests were equal to the average of the tip and butt ratios, the bending strength ratios are shown on Figures 33 and 34 by the triangular symbols. The poles tested by Thompson (1969b) were selected so that the largest knot in the lower 10 ft (3m) at the butt was 1 inch (25mm) or less and the pole diameter 6 feet (1.8m) above the butt was greater than 7.8 inches (198mm). Hence, the largest knot ratios for these poles were less than 0.13. In Figure 33 bending strength ratios for the poles tested by Thompson (1969b) are indicated by the square symbols at a knot ratio of 0.13. Arrows are attached to these symbols to indicate that the actual knot ratio may be less than 0.13.

While the bending strength ratios from the ASTM Pole Study (Wood, et. al, 1960) cannot be plotted in Figures 33 and 34 because the knot ratios were not reported, the approximate expressions recommended in the ASTM Pole Study (Wood, et al, 1960) for evaluating the strength reducing effects of the knot ratios given in equations 23 and 24 are shown in Figures 33 and 34. The data summarized in Figures 33 and 34, with the exception of the machine shaved Corsican pine poles, are closely approximated by the expressions proposed in the ASTM Pole Study (Wood, et. al 1960). This is particularly true for the sum of knots ratio, as shown in Figure 34. The machine-shaved specimens reported by Walford (1980) indicate strength losses consdiderably greater than those indicated by Equations 23 and 24. While the shaving may account for the reduced strength ratios, it should also be noted that Walford oriented the piles so that the knots were in a position to have the most adverse effect, a precaution not exercised in the other test programs.

The available data in Figures 33 and 34 are limited, and assumptions were necessary in order to plot some of the points. The need for additional research directed at the influence of knots on bending strength of round timbers is clearly evident. It is recommended that the approximate re lat ionships, Equations 23 and 24, proposed in the ASTM Pole Study (Wood, et al, 1960) be used to estimate the bending strength ratios associated with knots. In cases where major bending stresses must be resisted, it would appear prudent either to prohibit machine shaving, or to use a bending strength ratio equal to 80 percent of the strength ratio indicated by equations 23 and 24.

The extent of checks, shakes, and splits were not reported in detail in any of the test programs reviewed. The Wood Handbook (1974) and ASTM D245 indicate that shakes, checks, and splits have little or no effect on the strength properties in axial compression. The strength reduction associated with treatment, ψ , and the bending strength ratio associated with knots, ϕ b, indicated by the various test programs showed some trends toward slightly higher bending strength reductions for treated piles. This trend reflects in part the increased checking that accompanies the

TABLE 32 - BENDING STRENGTH RATIOS FROM CORSICAN PINE POLES (AFTER WALFORD, 1980)

		Knot Ratios[2]		
Test Series	Strength Ratios, ϕ_b [1]	Largest Knot, k_L/D	Sum of Knots, k_S/D	Notes[3]
A	0.95	0.15	0.20	hand peeled
B	0.94	0.17	0.20	hand peeled
C	0.73	0.17	0.23	machine shaved
E	0.60	0.18	0.24	machine shaved
F	0.76	0.16	0.23	machine shaved
G	0.78	0.14	0.19	machine shaved
H	0.73	0.15	0.22	machine shaved

[1] Ratio of average pole strength to average small clear strength
[2] Ratio of average knot dimension to average diameter
[3] All except series F were steam conditioned and treated

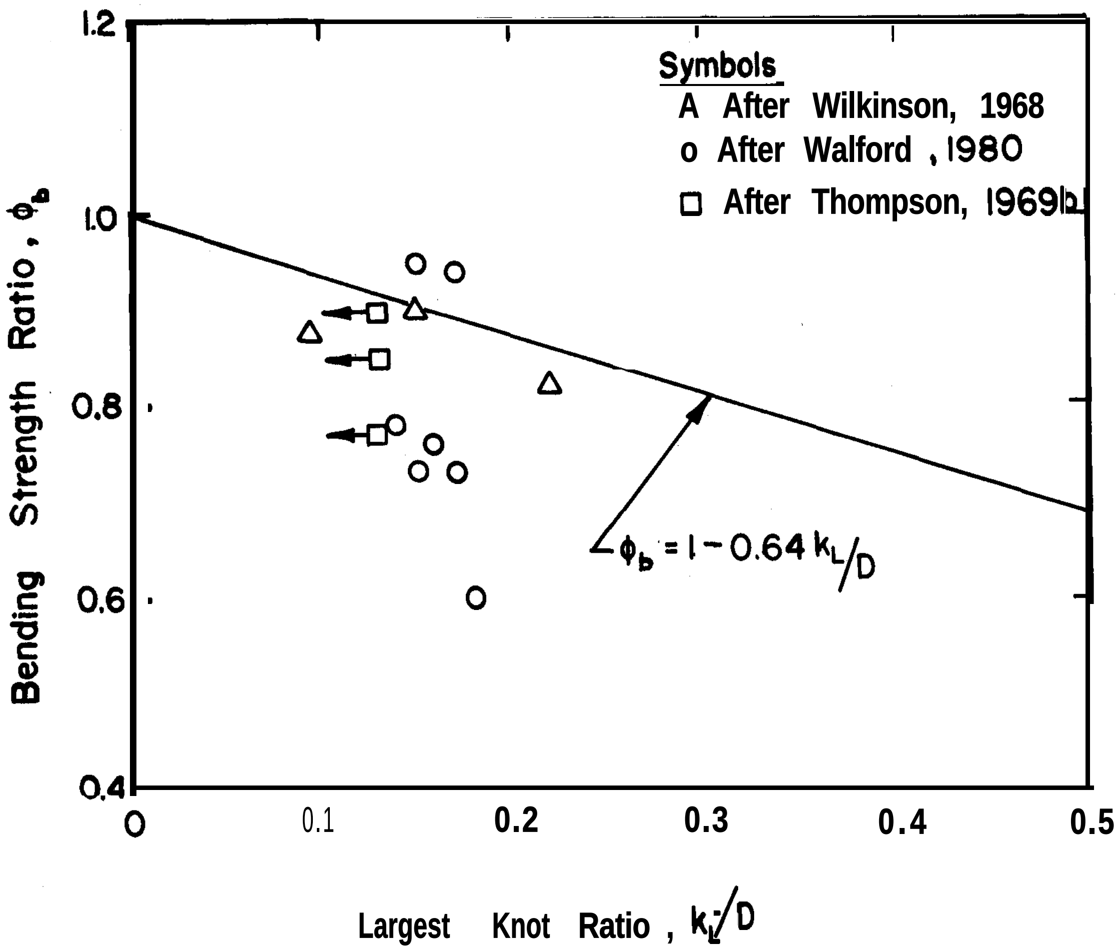


Figure 33. Influence of Largest Knot on Bending Strength Ratio

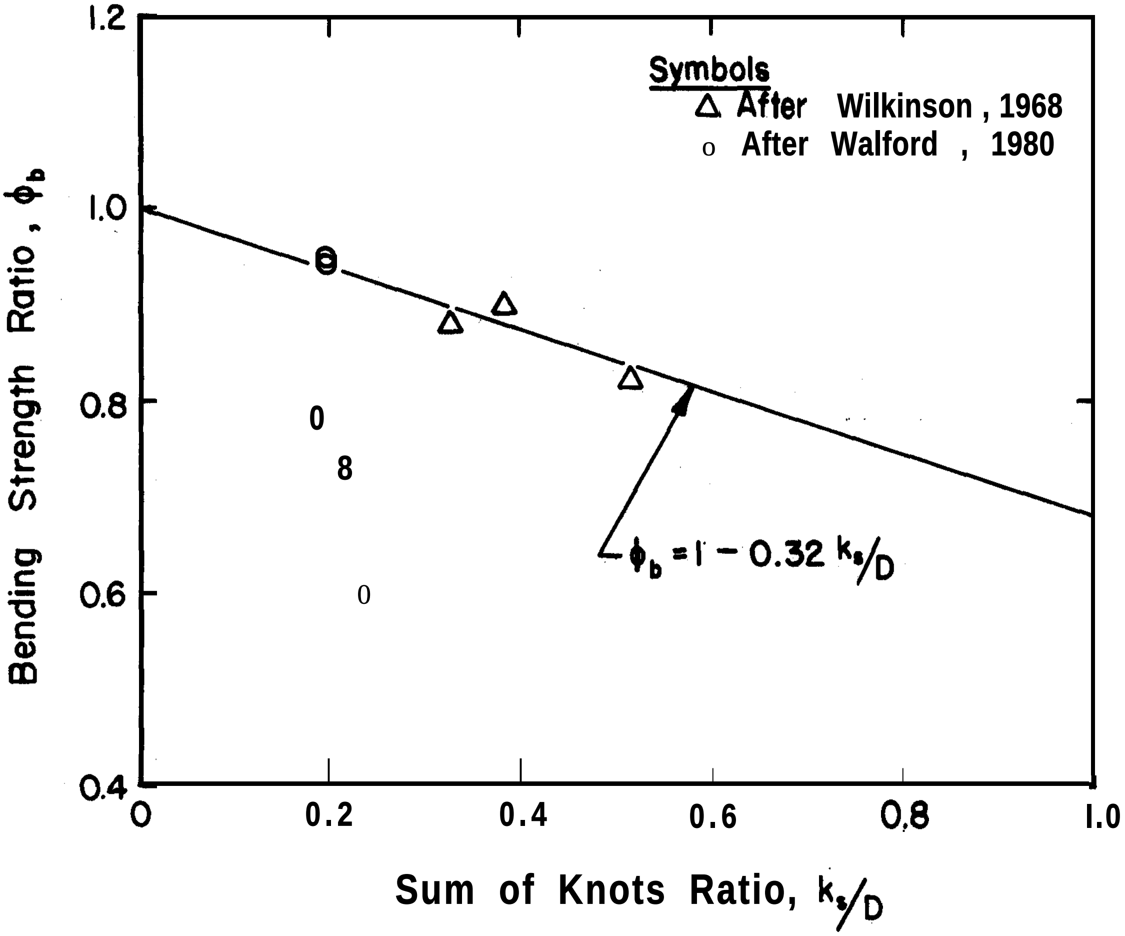


Figure 34. Influence of Sum of Knots on Bending Strength Ratio

conditioning process. For lack of quantitative data, it is assumed that the extent of imperfections such as checks, shakes, and splits that occurred in the test programs discussed herein encompasses the limits of these imperfections permitted under current materials standards, and that their effect is less than the strength reducing effects of knots. Since the majority of the strength ratios *were* from test programs on treated specimens, the adverse influence of treatment has been incorporated in the preceeding recommendations. The need for additional information on the strength reducing effects of checks and shakes in timber piles is clearly evident.

The Mississippi State University pile test program was the only test program that reported in detail the degree of spiral grain. (Thompson, undated and Thompson 1969a). Over two-thirds of the samples had grain slopes of 1 in 19 or less; therefore, the influence of spiral *or* sloping grain was minor. Localized grain distort ion around knots may have significantly influenced the strength. However this influence was inherently included in the correlations between the strength ratio and knot dimensions. In the absence of data on round timber members, it is recommended that the strength reductions used with sawn lumber, Table 15, be used to estimate the strength ratio for spiral or sloping grain in timber piles. Because most pile material specifications limit the spiral grain to less than 180° of twist over a distance of 20 feet (6.1m), the strength ratio associated with spiral grain will not control the allowable stress unless the critical section occurs at a location with a diameter greater than approximately 14 inches (356mm). Hence, except in those cases *where* bending stresses from lateral loading cause the critical section to occur near the butt, the influence of spiral grain within currently permitted limits can be ignored.

Based on the above review and recommendations on the influence of imperfections on timber pile strength, the strength ratios associated with the imperfections for both bending and compression are summarized in Table 33. The recommended strength ratios in Table 33 are based on the influence of knots indicated by Equations 23 through 26 and the permitted knot dimensions under AASHTO M-168 (Table 22). Where timber piles are used to resist large lateral loads, the resulting bending stresses combined with axial stresses can cause the critical section to occur near the pile butt. In such cases when the pile diameter at the critical section exceeds approximately 13 inches (330mm), the strength ratio should be taken as the value indicated in Table 33 *or* the value indicated by Table 15 for the permitted slope of grain, which ever is lowest.

The above recommendations assume that the piles are peeled or debarked in a manner that follows the natural contour of the pile. When piles are machine shaved to a smooth tapered cylindrical form, the wood removal at the knot swells can lead to a decrease in the strength ratio (Walford, 1980). Walford (1980) indicates that the bending strength of machine shaved poles is approximately 80 percent of the strength of peeled Poles. Qualitative evidence of reduced pile strength resulting from the machine shaving has also been discussed in meetings of ASTM Subcommittee D07.07 on Round Timbers (Arsenault, 1979). The machine shaving no doubt results in a strength *ratio* that lies somewhere between the upper bound limit of peeled piles and the lower bound of sawn lumber, but additional research is needed to quantify the influence of machine shaving. Unti 1 such inf ormat ion

TABLE 33 - STRENGTH RATIOS FOR AASHTO M-168 PILES [1][2]

Pile Length	Compression Strength Ratio, ϕ_c	Bending Strength Ratio, ϕ_b
For piles less than 50 ft. (15.2 m) in length	0.85	0.79
For piles greater than 50 ft. (15.2 m) in length		
(a) from butt to 3/4 of length below butt	0.85	0.79
(b) from tip to 1/4 of length above tip	0.78	0.68

[1] Based on imperfections permitted in AASHTO M-168 (See Table 22) and Equations 23 through 26.

[2] When pile diameter exceeds 13 inches (33.0 cm) at the critical section, the strength ratio shall be taken as the value above', or the value indicated for slope of grain in Table 15, which ever is lower.

becomes available, it would appear prudent either to restrict the use of shaving, or to estimate the strength ratio for shaved piles as 80 percent of the strength ratio for peeled piles.

Influence of Shape and Size on Timber Pile Strength, f - The bending strength of a member is dependent on both the size and shape of the member (Wood Handbook, 1974; Gurfmkel, 1981: and NFPA, 1977). For round members the modulus of rupture is approximately 1.18 times the modulus of rupture for square members. In sawn lumber the bending strength of rectangular members, as indicated by the modulus of rupture, decreases as the depth of the member increases (Bohannan, 1966). In the preceding section the bending strength ratio for the influence of imperfections was evaluated for the various test programs by.comparing the strength of round pile and Pole sections with diameters ranging from approximately 8 to 12 inches (200 to 300mm) to the strength of small clear specimens. Thus, the strength ratios incorporated the influence of shape and the influence of size up to approximately 12 inches (300mm). Hence, for pile diameters at the critical section of 12 inches (300mm) or less, the adjustment factor for shape and size, f, has a value of 1.0. For pile diameters greater than 12 inches (300mm), the adjustment factor, f, can be evaluated on the basis of Equation 27, where D is the pile diameter.

$$f = (12/D)^{1/9} \qquad \text{EQ. 27}$$

Because the critical section of most practical uses of timber piles occurs at a diameter less than 12 inches (300mm), the value of f will be taken as unity herein for developing allowable stresses for timber piles. In those special instances where the diameter at the critical section exceeds 12 inches (300mm), the allowable stresses should be reduced by the adjustment factor, f, given by Equation 27.

Variations of Strength within Timber Piles, Y - The majority of the strength and specific gratity data upon which the small clear wood strengths in ASTM D2555 are based, was obtained from samples 8 to 16 feet (2.4 to 4.9m) above the ground line (ASTM D143-52; Bohannan, 1971; and Wolf, 1978). While there are several studies of the variations of specific gravity and strength with height in a tree, definitive trends for all species or groups of species are not available. The available data suggest three types of clear wood strength variations with height may be observed; (1) decreasing strength with height, (2) increasing strength with height, and (3) an initial decreasing strength with height followed by an increase in strength near the crown (Okkonen, et. al, 1972; Panshin and De Zeeuw, 1980). In general, almost all data for the softwoods most frequently used for piling, Douglas firs and southern pines, show a trend for decreasing strength with height. For hardwoods and other softwoods the variation in strength with height are less predictable (Panshin and De Zeeuw, 1980; Wolf, 1978; Okkonen, et. al., 1972; Bohannan, 1971; and Wilkinson, 1968).

Bohannan (1971) reviewed the existing data on the relationship between bending strength and height for round timbers. Bohannan's summary of the available data is presented in Figure 35. With the exception of the two dashed lines which represent approximations based on observed specific gravity variations with height and specific gravity-strength correlations, the data in Figure 35 was derived from the results of pile and pole bending tests. Bohannan recommended the solid line on Figure 35 as a reasonable

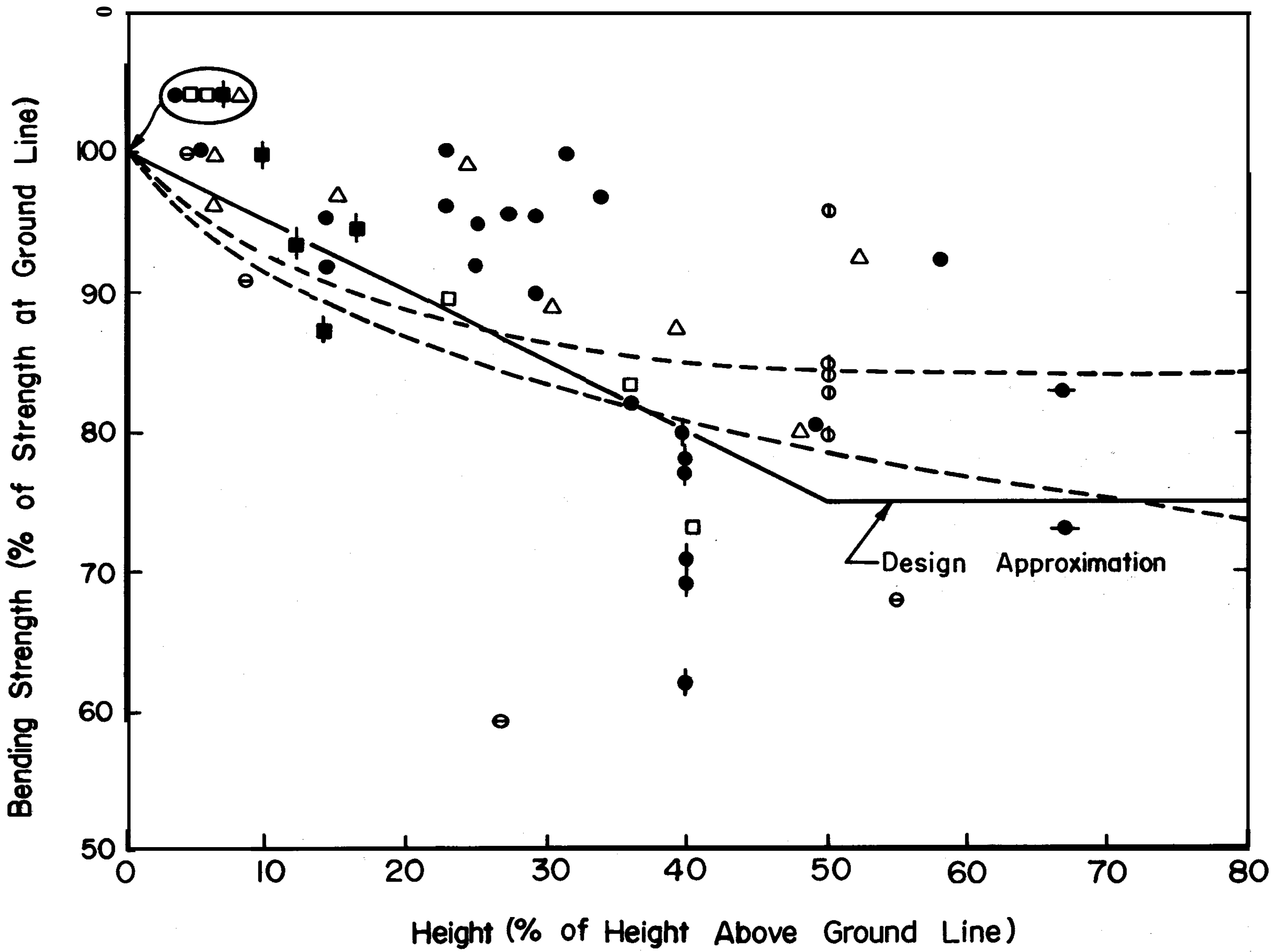


Figure 35 : Bending Strength - Height Relationship (After Bohannan, 1971)

design relationship between pole bending strength and height. The bending strength decrease with height for piles and poles reflects not only the decrease in clear wood strength with height, but also the decrease in section modulus and the trend for increase in knot size and frequency toward the pile or pole tip. Hence, Bohannan's recommended design value is probably an overly conservative estimate of the adjustment factor, γ, for clear wood strength variation between butt and tip.

Wilkinson (1968) reported results of a test program on 50 and 30 ft (15.2 and 9.1m) piles which included compression tests on both butt and tip specimens. Values of the adjustment factor γ, based on the ratio of the average tip small clear strength to the average butt small clear strength for Wilkinson's tests are summarized in Table 34. Wilkinson's tests are the only ones that allow a direct determination of the adjustment factor, γ.

Thompson (undated, 1969a) presented the results of small clear compression test specimens from 50 and 35 ft. (15.2 and 9.1m) southern pine piles. By comparing the average untreated small clear strengths to the average small clear strength indicated in ASTM D2555, Thompson's data can be used to estimate Y-values. The estimated γ-values range from 0.85 to 0.97, depending on the species assumed when selecting the ASTM D2555 strength.

While the available data on the variation of small clear strength from butt to tip is limited, a general trend for strength decrease from butt to tip is indicated for Douglas fir and southern pine piles. However, the decrease in clear wood strength with height is probably less than the bending strength reduction indicated for full pile and pole sections (see Figure 35). The only data on hard woods (see red oak species in Table 34) indicated an increase in strength from butt to tip. Excluding this hardwood sample and the γ-value of 0.66 for the steam treated southern pine sample reported by Wilkinson, a design value of 0.9 for the adjustment factor, γ, appears reasonable. Because of the limited data on species other than Douglas fir and southern pine, it is recommended that a γ-value of 0.9 be used for all species until further data becomes available. While this may be conservative for some species such as red oak, the lack of consistency in specific gravity-height variations for species other than southern pine and Douglas fir (Panshin and De Zeeuw, 1980 and Okkonen, et. al. 1972) indicate that this conservatism is warranted.

Factor of Safety for Developing Allowable Timber Pile Stresses, f_s - The strength ratio, ϕ, the adjustment factor for conditioning during treatment, ψ, and the adjustment factor for variation in strength from butt to tip, γ, are not single valued variables. They are actually multivalued, with a range or distribution much like the small clear wood strength, although their variation may not be as large. Hence, by applying these adjustment factors cumulatively, some margin of safety may be introduced in the process because the probability is low that the lowest value of the clear wood strength and the lowest values for the various adjustment factors will all occur simultaneously. In sawn lumber the use of near minimum adjustment factors applied to the 5 percent exclusion clear wood strength and a near minimum factor of safety of 1.25 to 1.5 is used in current design (Wood Handbook, 1974, and ASTM D245). The multivalued nature of the factor of safety in sawn lumber, resulting from the application of near minimum values of the various adjustment factors has been discussed by Wood (1958).

TABLE 34 - ADJUSTMENT FACTORS, γ, FOR STRENGTH DIFFERENCE BETWEEN BUTT AND TIP FROM WILKINSON'S TESTS (AFTER WILKINSON, 1968)

Species	Pile Length	Condition	Adjustment Factor, γ [1]
Douglas fir	50 ft. (15.2m)	Treated	0.89
Red oak	50 ft. (15.2m)	Treated	1.08
Southern pine	50 ft. (15.2m)	Treated	0.66
Southern pine	30 ft. (9.1m)	Green	0.91
Southern pine	30 ft. (9.1m)	Kiln-dried	0.85
Southern pine	30 ft. (9.1m)	Treated	0.90

[1] Ratio of average tip small clear strength to the average butt small clear strength

To evaluate the influence of the multivalued nature of the various adjustment factors, and the variation of clear wood strength, Wolfe (1978) and Randolph (1979) used a Monte Carlo simulation to model the pile strength distribution resulting from the random combination of clear wood strengths and adjustment factor distributions. These computer simulations indicated that the use of average values of the various adjustment factors and the 5 percent exclusion small clear wood strength results in a reasonably accurate estimate of the 5 percent exclusion pile strength. Hence, it would appear that the use of adjustment factors and strength ratios based on average tests values, a near minimum factor of safety as used in sawn lumber, and a 5 percent exclusion for small clear strength will result in reasonable allowable pile stresses, with a gross factor of safety approximately equal to that currently used in sawn lumber.

For timber piles a near minimum factor of safety, fs, of 1.25 in compression and 1.40 in bending is recommended for developing allowable stresses. These values were determined by using Equation 15 and the average values of the adjustment factor, F_{sd} for softwoods and hardwood for sawn lumber, Table 16. Considering the limited data available on the various reduction factors, the use of different fs-values for softwoods and hardwoods is unwarranted for timber piles.

Influence of Eccentricity on Allowable Axial 'Stress; ε - Because of the high probability of accidental eccentricity under practical pile installations, allowable axial pile stresses should include an allowance for a minimum eccentricity. For developing allowable stresses for timber piles a minimum eccentricity of 0.05 times the diameter will be used. This same value was used in the preceeding chapters on concrete and steel.

An expression for the adjustment factor for eccentricity, ε , resulting from an eccentrically applied force can be developed from the service load interaction formula for short columns given in equation 20. For an eccentricity of 0.05 times the diameter, the resulting expression for a round timber pile is given in Equation 28;

$$\varepsilon = 1/(1 + 0.4\ s_{ac}/s_{ab}) \qquad \text{EQ. 28}$$

where s_{ac} is the allowable concentric axial design stress and s_{ab} is the allowable bending design stress. Substitution of the recommended values for the adjustment factors f_s γ , ψ , β , ϕ , and f; and Equations 21 and 22 into equation 28 results in the following expression.

$$\varepsilon = 1/(1 + 0.5\ s'_c/s'_b) \qquad \text{EQ. 29}$$

The stresses s'_c and s'_b are the 5 percent exclusion values for the green small clear crushing strength parallel to grain and the green small clear modulus of rupture, respectively. The 5 percent exclusion strengths in Tables 7 and 8 for Douglas firs, southern pines, and red oak species results in e-values ranging from 0.80 to 0.84 with an average'value 0.82. For the purpose of deriving allowable design stresses a ε -value of 0.82 will be used for all species.

Summary of Recommended Strength Ratio and Adjustment Factors for Round Timber Piles - In the preceeding sections, available information on the various strength ratios and adjustment factors were reviewed and design values were recommended. The recommended design values for the various factors are summarized in Table 35 . By substituting the values of the various factors into Equations 21 and 22, the ideal (HDF = 1.0) allowable structural design stresses for round timbers can be expressed as a fraction of the 5 percent exclusion value for the green small clear strength property, s' . The recommended design values for compression and bending in round timbers under ideal conditions, HDF = 1.0, are summarized in Tables 36 and 37, respectively.

Moment-Thrust Interaction Diagrams for Embedded Piles - When timber piles are subjected to combined axial and bending forces, the effective eccentricity may exceed the minimum design eccentricity of 0.05d included in the allowable compression stress values in Table 36. In such instances, the combined axial thrust and bending in embedded timber piles can be addressed by the use of the service load interaction formula given in Equation 20. Equation 20, when modified to be expressed in terms of the allowable stresses in Tables 36 and 37, results in Equation 30:

$$(0.82f_c/s_{ac}) + (f_b/s_{ab}) = 1.0 \qquad \text{EQ.30}$$

where:

f_c = the axial compression stress
f_b = the bending stress
s_{ac} = allowable compression stress from Table 36
s_{ab} = allowable bending stress from Table 37

Because of the tapered nature of timber piles, it is generally more expedient to use the interaction formula in Equation 30 rather than constructing moment-thrust interaction diagrams. However the moment-thrust interaction diagram at a particular section can be constructed easily from the values in Tables 36 and 37. The allowable concentric axial load, P_o, is given by Equation 31; where A is the cross-sectional area and s_{ac} is the allowable axial stress from Table 36.

$$P_o = (s_{ac}A)/0.82 \qquad \text{EQ. 31}$$

The allowable bending moment at zero axial load, M_o, is given by Equation 32; where S is the section modulus and s_{ab} is the allowable bending stress from Table 32.

$$M_o = s_{ab}S \qquad \text{EQ.32}$$

The service load interaction diagram can then be constructed by drawing a straight line between P_o and M_o. In 'Figure 36 a design service load interaction diagram for a round timber pile is illustrated by the solid line. The value of the horizontal segment, P_a, corresponds to the minimum design eccentricity of 0.05D. Alternatively the value of P_a, can be estimated by Equation 33.

$$P_a = s_{ac}A \qquad \text{EQ.33}$$

Driving *Stresses - For efficient driving a's well as the efficient utilization of pile material, 'it is generally desirable to stress the pile to the practical limit during driving. Hence, when evaluating driving

TABLE 35: SUMMARY OF RECOMENDED STRENGTH RATIOS AND ADJUSTMENT FACTORS FOR ROUND TIMBER PILES

ijustment	Symbol	Recommended value
›ad Duration	β	■ 0.625 for normal load duration ■ 0.5625 for long term loading ■ 0.83 for load tert duration ■ 1.2 for analysis of driving stresses (Note: in ideal conditions a value up to 1.5 might be attained) For other durations ree Table 17
onditioning Process uring Treatment	ψ	■ 1.0 for untreated or air seasone piles ■ 0.9 for kiln drying ■ 0.85 for the Boulton process (heating in the preaervat ive) ■ 0.75 for steaming
mperfections	ϕ	In Compression: ■ 0.85 for piles less than 50 ft in length and the upper 3/4 of piles greater than 50 ft. ■ 0.78 for the tip 1/4 of the length of piles greater than 50 ft. In Bending: ■ 0.79 for piles less than 50 ft in length, and the upper 3/4 o piles greater than 50 ft. ■ 0.68 for the tip 1/4 of the length of piles greater than 50 ft.
ihape and Size	f	■ 1.0 for pile diameters of 12 incheo or less ■ $(12/D)^{(1/9)}$ for pile diameters greater than 12 inches
location in Pile	γ	■ 1.0 at butt ■ 0.9 at tip
tinimum Design	ε	■ 0.82 (based on minimum eccentricity of 0.05 times the diameter)
Factor of Safety	f_s	■ 1.25 in compression ■ 1.40 in bending

Note: 1 ft.. ■ 305mm; 1 inch ■ 25.4mm.

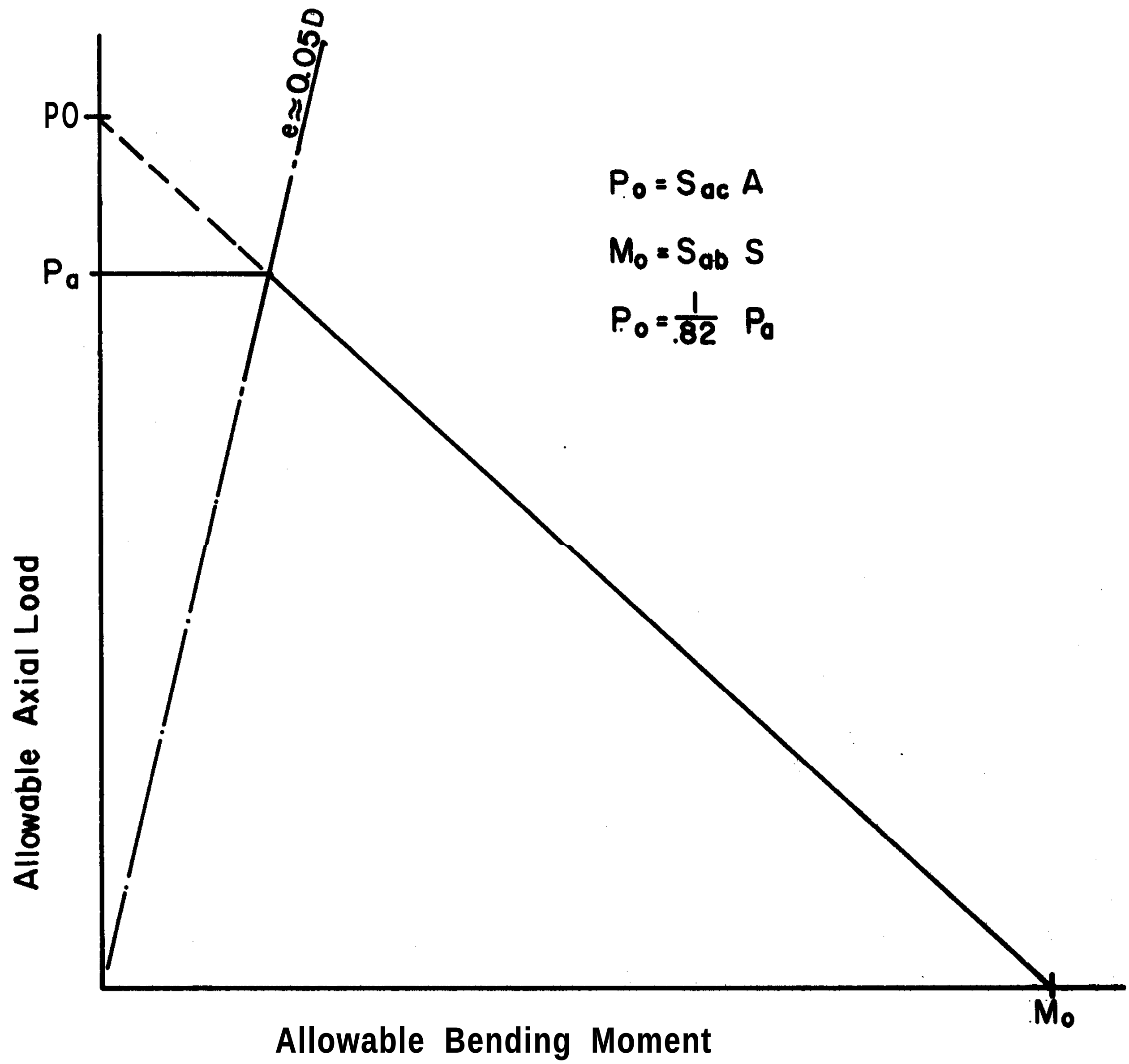


Figure 36. Moment-Thrust Interaction Diagram for Timber Piles

stresses, it is desirable to deal with ultimate strength rather than allowable design stresses for static conditions. In addition, the important influence of load duration must be considered when evaluating driving stresses in timber piles. In the preceeding discussion on the influence of load duration on wood strength, it was indicated that a B-value of 1.2 to 1.5 could be used for evaluating driving stresses. Using these values, the 5 percent exclusion strength of a pile section under a concentric blow during driving is approximately 3.0 to 3.7 times the allowable stresses given in Table 36. Until additional information on the behavior of wood under repeated impact as experienced during pile driving becomes available, it is recommended that driving stresses be limited to $3s_{ac}$, where s_{ac} is the allowable design stress from Table 36.

RECOMMENDED ALLOWABLE DESIGN STRESSES FOR TIMBER PILES.

The preceding discussion of allowable stresses was related to the structural strength of a round timber member where the assumption of a short-column is applicable. The lateral soil resistance in even the weakest soils is sufficient to make the short-column condition applicable. However, piles are subjected to potentially destructive forces during driving, primarily the forces developed by the hammer and the induced soil/rock resisting forces. Further, almost all piles are driven somewhat out-of-plumb and have insitu curvatures, which results in a residual stress condition in the piles prior to the application of the structural loads. In addition, under the most severe installation conditions, the structural integrity of the pile can be impaired, as illustrated in Figure 5 of Chapter 3. Therefore, it would appear prudent to apply a strength reduction factor to driven piles that is larger than the strength reduction applied to above ground members that are observable and maintainable.

In Chapters Four and Five this larger strength reduction was achieved by a separate reduction factor referred to as the hidden defect factor, HDF, the value of which was dependent on the pile type, subsoil conditions, and the inspectability of the pile after driving. For timber piles the potential for damage and the inspectability are almost identical to the H-pile? and the proposed values for the hidden defect factor are the same under ideal (1.0) and normal (0.85) conditions. However, because of the much lower tip strength of timber piles caused by strength decreases from butt to tip, by decreasing pile cross-sectional area, and by increases in both knot size and frequency, the potential for structural damage during driving is so large that the use of timber piles may become impractical under severe subsoil conditions. Hence, rather than providing a value for the hidden defect factor under severe conditions it is recommended that the use of timber piles under severe conditions be prohibited, unless actual field driving and extraction tests verify that the timber piles can be installed on the particular site without impairing the required structural capacity. The recommended allowable design stresses for timber piles can be developed by multiplying the ideal allowable structural stresses in Tables 36 and 37 by the appropriate hidden defect factor, HDF.

ANALYSIS OF TIMBER INDUSTRY RECOMMENDATIONS

The timber industry's lobbying efforts for higher allowable stresses

TABLE 36 - RECOMMENDED ALLOWABLE COMPRESSIVE STRESSES, s_{ac} [1] [2] [5]

Location	Pile Length	Treatment Conditioning [3] [4]: Untreated or Air Seasoned	Kiln dried	Boulton Process	Steamed
Pile Butt	All lengths	$0.35s'_c$	$0.31s'_c$	$0.30s'_c$	$0.26s'_c$
Pile Tips	50 ft. (15.2m) or less	$0.31s'_c$	$0.28s'_c$	$0.27s'_c$	$0.24s'_c$
	over 50 ft. (15.2m)	$0.29s'_c$	$0.26s'_c$	$0.24s'_c$	$0.22s'_c$

[1] For normal load duration
[2] Assumes piles are furnished according to quality standard AASHTO M168
[3] Assumes treatment under AASHTO Specification M133
[4] s'_c = 5% exclusion limit in compression parallel to grain for green small clear wood specimens
[5] Includes a reduction of 0.82 for a minimum design eccentricity of 0.05 times the diameter

TABLE 37 - RECOMMENDED ALLOWABLE BENDING STRESSES, s_{ab} [1] [2]

Location	Pile Length	Treatment Conditioning [3] [4] [5]: Untreated or Air Seasoned	Kiln dried	Boulton Process	Steamed
Pile Butt	All lengths	$0.35s'_b$	$0.32s'_b$	$0.30s'_b$	$0.26s'_b$
Pile Tips Tips	50 ft. (15.2m) or less	$0.32s'_b$	$0.29s'_b$	$0.27s'_b$	$0.24s'_b$
	over 50 ft. (15.2m)	$0.27s'_b$	$0.25s'_b$	$0.23s'_b$	$0.20s'_b$

[1] For normal load duration
[2] Assumes piles are furnished according to quality standard AASHTO M168
[3] Assumes treatment under AASHTO Specification M133
[4] s'_b = 5% exclusion limit for modulus of rupture of green Small Clear wood specimens
[5] For diameters over 12 inches multiply by $\left[\frac{12}{D}\right]^{1/9}$

are reflected in ASTM D2899-74, Standard Method for Establishing Design Stresses for Round Timber Piles. No justification has been published for this standard. However, considerable information has been assembled on the strength of timber piles by Armstrong (1979), and the complete discussion given in this chapter allows a succinct evaluation to be made. The express ion:

$$s_a = \phi \gamma \psi \beta (ecc)(HDF)(s'_c)/FS$$

will be used as a basis for comparison.

The values for the various factors in the above equation are given in Table 38 for ASTM D2899-74, and compared to those recommended herein for timber piles meeting the AASHTO specification. ASTM D2899-74 is too liberal on the strength ratio ϕ, the treatment factor ψ , and the factor Of safety, which they consider optional. Further, ASTM D2899-74 does not consider strength 'reductions due to eccentricity and hidden damage.

The factors given in Table 38 lead to allowable stresses that are tabulated in Table 39 for comparison with the allowable stresses from Section 1.44 of the 1977 AASHTO specification. The recommendations given in this report lead to allowable stresses approximately one-half of those currently allowed. This would appear at odds with precedence except for the fact that timber piles are generally not used for loads exceeding 25 to 30 tons (222 to 267 kN). AASHTO (1977) limits timber pile loads to 24 tons (213.5 kN) where the butt diameter is 12 inches (305 mm), Therefore, the recommended stresses have seldom been exceeded in practice, and precedence is not applicable.

TABLE 38 - REDUCTION FACTORS FOR COMPRESSION PARALLRL TO GRAIN FOR TIMBER PILES

Factor	ASTM D2899-74	This Report
ϕ - Strength Ratio	0.90	0.78 - 0.85
γ - Adjustment butt to tip	0.90	0.90
ψ - Adjustment for treatment ,	0.90 (Boulton) 0.85 (Steam)	0.85 (Boulton) 0.75 (Steam)
β - Adjustment for load duration	0.66	0.625 (Normal) 0.5625 (permanent)
ecc - Eccentricity factor	1.0	0.82
HDF - Hidden Damage Factor	1.0	1.0 & 0.85
s'_c - 5% Exclusion - small clear	ASTM D2555	ASTM D2555
FS - Near minimum factor of safety	1.25 optional	1.25

TABLE 39 - TIMBER PILE TIP ALLOWABLE STRESSES, IDEAL CONDITIONS

Type of Pile	This Report for AASHTO M168[1]	AASHTO 1977 Sec. 1.4.42	ASTM D2899-74 for ASTM D25-79
Southern pine steamed	525-600	1200-1400	1200
Douglas rir boulton	600-700	1100-1200	1250

[1] Allowable stress depends on pile length.
[2] Allowable stress depends on species or grading.

CHAPTER SEVEN
ENVIRONMENTAL FACTORS AND PILE DAMAGE

INTRODUCTION

Each of the three pile materials discussed herein are subject to reduction of strength due to corrosion and/or deterioration. The causes of corrosion and deterioration are usually related to the environment in which the piles exist. Causes of pile deterioration, for example, range from fungi, insect attack and marine borers in the case of timber to sulphate attack on concrete, and to corrosion of steel. Abrasion of piles by water borne sediments and other solids has also been reported. Another process that is mechanical in nature is freeze-thaw deterioration of concrete. These processes damage the pile section by reducing the effective cross-sectional area, or otherwise weakening it. Mechanical damage to the Pile during driving can be added to the above mentioned causes of damage. All piles can be damaged by using too-large a hammer for the pile; however, the subject of matching hammer to pile is beyond the scope of this report. Pile damage considered herein due to driving is that which occurs even though the hammer is properly matched to the pile. The cause of such damage resides in the nature of the soil/rock conditions into which the pile is driven.

It is necessary for the structural designer to consider the anticipated life of the structure he is designing. The designer must provide the requisite strength in a pile foundation at all times during the life of the structure. The allowable stresses that were derived in Chapters Four through Six make no allowance for corrosion, deterioration, abrasion, fungi or insect attack, etc. ; however, uncomtemplated damage due to driving was considered by use of the hidden defect factor, HDF. Thus, the designer has the obligation to provide protection for the load carrying capacity of the pile insofar as environmental factors other than pile damage may influence it. This is a difficult assignment for the designer because each project site has a unique set of environmental conditions.

Methods that have been used by designers in the past to protect against loss of capacity because of corrosion/deterioration consist of:

1. Deducting thickness from the perimeter of the material before calculating the strength of the section; e.g., deducting 1/16 inch (1.6mm) from each exposed surface of a steel member.
2. Using a relatively low allowable stress and assuming that the resulting oversize structural member has sufficient reserve material to accommodate corrosion/deterioration.
3. Provide a coating for the piles or other protective measures, such as cathodic protection.
4. Providing for repair and maintenance of the piles.
5. Using a type of pile that is not susceptible, or is the least susceptible, to deterioration.

An overview of corrosion/deterioration and pile damage is given for each of the three pile materials in the following sections of this chapter.

TIMBER

Deterioration

Decay of timber and the attack by marine borers is a well known phenomenon. Timber was the first material to be used extensively for piling; hence, there is a long experience record with it. A lengthy discussion of deterioration and preservation of timber piles is given by Chellis (1961). This reference is widely used and is available in most structural engineers' libraries; for this reason, only the essential conclusions that can be drawn from Chellis (1961) are presented.

The principal factors which cause timber piles to deteriorate are:

1. Decay
2. Insect Attack
3. Marine-Borer Attack
4. Mechanical wear
5. Fire

Decay is caused by fungi that need air and some moisture to survive. Thus, timber below the permanent water table is naturally preserved. Timber above the permanent water table, however, is susceptible to decay depending on the availability of air. There are case histories (Chellis, 1961) where the water table has been lowered after a structure on untreated timber piles has been in place, and the piles then deteriorated. Termite and beetle attacks also take place above the water table. Marine borers, on the other hand, attack piling from the mudline upwards.

Damage from decay, insect attack and marine borers can either be prevented entirely or the life of a timber pile prolonged by preservative treatment. Foundation piles in the United States are usually treated with creosote, which is toxic to fungi, insects and borers. Other treatment chemicals may receive more use in the future because of changing economics.

Pile damage caused by mechanical wear (abrasion) and fire is of interest primarily in trestles and waterfront construction. The fire hazard is usually accepted as a risk. Mechanical wear, on the other hand, can be resisted by replaceable or semi-permanent armor coating (Chellis, 1961).

Composite piles are sometimes used to avoid deterioration problems with timber. A composite pile frequently used in Louisiana consists of an untreated timber lower section and a mandrel-driven steel corrugated shell upper section. The timber is driven entirely below the permanent water table and the upper steel shell is filled with concrete, Pile load capacity is controlled by the weakest of the two materials, namely, the timber.

With the exception of the composite pile described above it has long been established that timber foundation piles shall be treated with one or more preservatives. The difference in cost between treated and untreated timber has seldom been judged to justify the risk. A review of the characteristics of timber piles and case histories of failures (Chellis, 1961) is ample support for the cost of preservative treatment.

Driving Damage - Damage to the lower portions, primarily the tip of timber piles during driving is well known to experienced foundation engineers. Such damage was mentioned in Chapter Three, and photographs of damaged piles are shown in Figure 5. A stress analysis of the driving of timber piles was carried out by Davisson (1975). The results confirmed what had been observed over many years of experience. Because of the taper of timber piles, the downward force of driving is resisted by less cross-sectional area of pile as *the* force wave travels downward. As a consequence, the tip is highly stressed and, therefore, more easily broken than the rest of the pile.

Piles that resist driving primarily at the tip should be driven with a small hammer to avoid breakage e.g. a #2 Vulcan with a rated energy of 7250 ft-lb (9830 joules) per blow. *Where* the pile tip size is 8 to 9 inches (200 to 230 mm) or larger, it may be possible to use a larger hammer, such as the #1 Vulcan rated at 15,000 ft-lb (20,337 joules) per blow. However, in neither case should driving resistance exceed 5 blows/inch (25 mm) at any depth during driving.

For piles that resist driving primarily by skin friction, the stress conditions are much more favorable because of attenuation of the driving force by the friction. Further, compressive wave reflections at the tip are either small or non-existent. It is in such conditions that piles can be driven the hardest against the soil, thereby developing the highest soil-pile bearing capacity. Hammers as large as a Vulcan #06 rated at 19,500 fe-lb (26,438 joules) can be used.

In ehe study referred to above (Davisson, 1975) it was also found that the McDermid driving base was undesirable, and the driveheads incorporating hammer cushions of wood or equal seiffness were preferred. Diesel hammers were also found satisfactory for driving timber piles. The rated sizes of diesel hammers could be 30 eo 50 percent higher than for air/steam hammers; hammer cushions should be per manufacturer's recommendations.

The foregoing discussion provides guidance on pile driving equipment and final driving criteria that should help prevene damage. It must be recognized, however, that soil conditions were assumed to be ideal. If Cobbles, boulders or other obstructions are encountered that deflect the pile tip during driving, stress conditions will be set up that are likely to break the pile tip. The same would be true if an uneven bedrock surface is encountered. These conditions are severe with respect to timber piles. Considering that the embedded port ions of timber piles are uninspectable, it seems prudent not to use timber piles in severe conditions.

CONCRETE

Durability - The subject of concrete durability is thoroughly covered in most modern textbooks (Neville, 1973; Mindess and Young, 1981). Also, ACI Committee 201 has published a document "Guide to Durable Concrete", (ACI 201.2R-77) that contains an extensive reference list. Considering such thorough and readily available literature, this report will only present an overview of the factors pertaining eo ehe durability of concrete piles.

Concrete piles may suffer deterioration from the following causes:

1. Freeze-Thaw Cycles
2. Chemical At tack
   Aggregate Chemical Reactions
2: Steel Corrosion
5. Abrasion

Freeze-thaw will be of concern primarily in the case of free-standing portions of piles. Fully embedded piles will not generally be affected. The effect of freezing concrete pore water is expansion that cracks the cement bond, thus promoting deterioration. The mechanism is thwarted by using a strong impermeable concrete. This is accomplished by using a low water/cement ratio and air entrainment.

Chemical attack is of concern primarily in the case of sulfates and acid attack. Concrete piles are susceptible to such attacks in environments where acid or sulfate solutions are present. In general, the actions to counter chemical attack are to use a dense high quality concrete with a low water/cement ratio. Also, a sulfate resistant cement may be used (Type II or V). A coating should be used where these measures are insufficient .

Chemical attack on concrete piles is most likely to occur at those points along the pile where it penetrates soils that are free-draining. In such soils it is possible for ground water movements to replenish the supply of acid or sulfate solutions. In relatively impermeable soils the action of acid or sulfate solutions may be limited to the concrete cover or surface area of the pile because significant replenishment cannot occur. The zone above the water table should be considered as a zone of replenishment because of migration of surface water down to the water table.

Chemical reactions of aggregates may cause concrete deterioration. This is a problem that should be recognized during the mix design.

Corros'ion of reinforcing steel in a concrete pile can spa11 the cover because the products of corrosion are expansive. Corrosion of steel inside concrete is naturally inhibited (Mindess and young, 1981) unless cracks, high permeability, and thin cover permit corroding agents to contact the steel. Low concrete permeability is desirable and is obtained by using dense well graded aggregate, air entrainment, and a low water/cement ratio. Normally, 1 1/2 to 2 inches (38-51 mm) of cover is considered minimum. A cover of 3 inches (70 mm) is considered desirable in sea water. Epoxy coated reinforcing bars can be used to inhibit corrosion damage. Where the foregoing is inadequate, waterproof coating on the surface of the pile is required.

Concrete abrasion resistance is promoted by a strong concrete (low water/cement ratio), resistant aggregate, and the minimum air content consistent with the conditions of exposure. Two other techniques include providing a replaceable shield, or maintenance involving patching the abraded zone.

Some of the desirable characteristics of durable concrete are incompatible with those needed for piling, especially cast-in-place and cast-in-situ piling. In particular, a low water/cement ratio and a low slump cause workability problems.! Such concrete is readily handled in precast piling, but may promote voids in cased cast-in-place and cast-in-situ piling.

For cased cast-in-place piling there is an outer metal casing that protects the concrete until the casing is lost to corrosion. This either prevents or mitigates the effect of using high slump concrete. For cast-in-situ piles no casing is present, however, the ϕ -factors used in developing allowable stresses for cast-in-situ piles result in relatively larger cross-section for a given pile load when compared to other piles. This can be viewed as providing extra cover. In either case, it is recommended herein that high slump concrete be used as required to construct the pile, and if the resulting product does not have the desired durability, that other measures be adopted.

Driving Damage - It is assumed in this discussion that the driving equipment used for precast piles is designed and operated to meet the stress limitations given in Chapter Five. This should virtually eliminate driving damage caused by general overstress, both in tension and compression. Other damage may occur during driving, and the primary causes are eccentric contact of the pile with an obstruction such as bedrock or a boulder, or a sweep or dogleg in the pile that induces flexural cracking. Eccentric contact of the pile with bedrock can spall part of the tip or cause flexural cracking, or both. Thus, where such conditions are anticipated it is prudent to design tip reinforcement for the pile.

Cased cast-in-place piles do not involve driving damage to the concrete itself. Further, such piles are internally inspectable and can be rejected or down-graded as required to be consistent with the as-driven condition of the shell.

Cast-in-situ piles are not internally inspectable, and successful placement depends on skilled, experienced and interested workmen. This factor overshadows all other factors combined when considering the integrity of cast-in-situ piles. If the designer cannot be assured of proper workmanship, then such piles are not recommended for bridge foundations.

It should be noted that the allowable stresses given in Chapter Five account for some pile damage in the Hidden Defect Factor, HDF. However, this does not mean that the pile designer does not have to give further consideration to damage. Whatever forces the designer envisions should be accounted for in design. HDF accounts only for unanticipated defects.

STEEL

Corrosion - Extensive literature on the subject of pile corrosion exists. A summary of studies performed in North America is given in papers by Romanoff (1962) and the National Bureau of Standards (1972). More recently, the American Iron and Steel Institute (1982) has summarized the steel industry's views. Considerable work has also been done in Australia by Eadie (1977). The aforementioned work applies to fully embedded piling. A very large body of literature exists on the corrosion of exposed piling, and summaries have been made, for example, by Chellis (1961).

It is well known that steel rusts; the questions with respect to piling area: (1) how much, (2) under what conditions, (3) at what rate, and (4) what can be done to retard it. An example described by Sudrabin (1963) is pictured in Figure 6; the pile cross-section was diminished by more than 50 percent in a few years. Only a minor extrapolation is required to arrive at the conclusion that steel corrosion can lead to a total loss of section; only, the rate of corrosion is in question, and that varies widely according to the environment.

The concepts put forth by Romanoff (1962) appear to be accepted currently. It has been observed that corrosion is minimal or non-existent in undisturbed soil below the water table where oxygen is not available. An exception occurs if the pile penetrates a free-draining soil layer where water movement takes place. Under such conditions corrosion can occur because of a continuing supply of oxygen even though the concentration is quite low.

In the zone at the water table and above, oxygen can reach the steel and corrosion occurs. The rate at which corrosion occurs depends on the supply of oxygen and the chemical nature of the ground and the moisture migrating in this zone. There is some controversy over whether or not the rate of corrosion can be correlated with soil resistivity, pH and chemical composition. However, it is prudent to gather this information whenever it is suspected that corrosion may be eignif icant.

Two methods of protecting steel piles are currently in common use, namely, cathodic protection and epoxy coating, In the past it was common practice to deduct 1/16 in (1.6 mm) from each exposed face of a steel pile as being lost to corrosion in normal (non sea water) environments. A review of corrosion rates quoted in the literature (Chellis, 1961) indicates that this was a reasonable estimate. The most recent work in Australia (Kinson, Lloyd and Eadie, 1981) recommends that thickness allowance be made for corrosion. On the other hand, it can be argued that the allowable stresses commonly used (9,000 - 12,000 psi; 62-83MPa) result in pile cross-sections of sufficient size that some section can be lost to corrosion without impairing the required structural strength.

Pile Damage - Examples of damage to steel piles are given in Figure 4 of Chapter Three, and by Chellis (1961). Such damage often occurs even though the pile hammer and hammer cushion were designed to prevent a general case of damage in ideal soil conditions. The damage cited and discussed herein is caused by obstructions that usually initiate bending of the flanges of H-piles, The literature is thick with examples of damaged H-piles. Therefore, for non-ideal soil conditions it is prudent to require reinforced pile tips.

In the case of pipe piles in non-ideal soil conditions two conditions can be defined. Open-end pipe is not internally inspectable and is subject to undetected damage. Hence, in non-ideal soil conditions it is prudent to reinforce the tip. Internal inspection is possible for closed-end pipe; in this case, damage may be treated or corrected according to conditions.

As with timber and concrete, the hidden defect factor, HDF, accounts for undetected damage that occurs even when the designer has taken precautions to add reinforcement, tips, etc. for non-ideal soil conditions, HDF is not a substitute for the factors discussed herein.

CHAPTER EIGHT
PROCEDURE FOR HIGHER ALLOWABLE STRESSES

NEED

An engineering evaluation of a particular design and the site conditions may lead to the conclusion that the factors used to derive allowable stresses in Chapters Four through Six are not strictly applicable. Such an evaluation might indicate that a more conservative design or a more liberal design is justified. Be cause c ode provisions usually cover conditions where conservatism should be applied, the emphasis herein will be on conditions where a liberal design may be warranted. There is therefore a need for a procedure that leads to a knowledgeable selection of a higher design stress.

HIGH STRENGTH PILE MATERIALS

Probably the easiest means of increasing a structural design stress is to employ a material with a higher basic strength. For example, a grade 50 steel (F_y=50ksi, 345MPa) may be substituted for a grade 36 steel (F_y = 36ksi, 248 MPa), or concrete with a higher value of f'_c may be employed. Both steel and concrete are subject to manufacturing control by the engineer and the resulting strength generally can be kept within known bounds. On the other hand, timber is a natural material that varies along the length of a given tree, from tree to tree, and from species to species. Substitution of higher strength timber in a pile is not a simple task.

Whenever high strength pile materials and higher allowable stresses are substituted for those normally used, there is a decrease of pile cross-section used to support a given load. If this process is carried virtually to the limit, the pile would eventually become a very high strength nail that is driven into the ground with a very large hammer. It is obvious that a point would be reached where the pile (nail) would buckle under the hammer, and driving could not be accomplished. However, even before the point at which buckling occurs there are ranges of pile allowable stresses that result in selection of a pile cross-section that has insufficient axial stiffness to be driven against the soil with sufficient force to develop the required bearing capacity relative to the soil. This phenomenon was also described and/or discussed in Chapters Three through Six. For this reason it is prudent to drive and successfully load test a pile designed to the proposed higher allowable stress before construct ion begins, and preferably during the foundation design stage.

Other items to be considered when substituting a high strength pile material are corrosion and deterioration. Corrosion and deterioration are likely to penetrate the same for both normal and high strength materials (unless a corrosion resistant material is selected). In such an instance the high strength material has less reserve cross sectional area for sustaining the effects of corrosion and deterioration, Problems such as weldability of high strength steels, and the bearing stress of the piles on the pile caps may also become increasingly important.

HIGHER ALLOWABLE AS A PERCENTAGE OF STRENGTH

Steel - The expression for allowable stress (fa) given in Chapter Four can be analyzed for the purpose of assessing the effect of increasing the allowable stress.

$$fa = \phi\,(ecc)\,(HDF)\,(Fy)/LF$$

The ϕ -factor is necessary to arrive at the reliable strength of the material. A value of 0.85 is necessary until the specifications governing purchase, tolerances, and acceptance testing are changed. It is possible, however, to select pile members for which the apparent ϕ is 1.0 or higher. This is possible because in normal practice at the mill the yield strength of the heat can be adjusted by the metallurgist to a value that produces an apparent ϕ -factor of 1.0, although the mills are not required by specification to do this. Although the basic mill coupon strength is increased, the factors causing a ϕ -factor to exist have not been eliminated; hence, the term "apparent ϕ" is used. Similarly, a single pile load test, or several pile load tests, also fail to serve as proof that ϕ is higher than 0.85 because of the aforementioned mill practice. Therefore, the O-factor cannot, at this time, be changed to justify a higher allowable stress.

The eccentricity factor covers several strength reducing items. It properly penalizes pile cross-sections that are sensitive to eccentricity. Eccentricity may come from eccentric load application relative to the theoretical center of gravity of the pile cross-section, or theoretically concentric load application on a pile cross-section manufactured with unequal thickness of flanges or variable pipe wall thickness, or both. The effect of eccentricity can be minimized or eliminated in a pile load test. Thus, successful pile load tests cannot serve as proof that the eccentricity factor can be reduced or eliminated.

Of the remaining factors in the expression for allowable stress, the hidden defect factor (HDF) is likely to be assumed as unity indicating that soil conditions are ideal and will not damage the pile section. Otherwise, seeking a higher allowable stress would seem to be a questionable engineering judgement. The yield strength (Fy) has been discussed previously as a method of attaining a higher allowable structural stress; one can merely substitute a higher strength steel. Finally, the load factor appears to be where the effect of a higher allowable stress is felt. This is logical - higher stresses for a given material mean that the load factor, or the capacity to sustain an overload is reduced. The discussion that follows indicates many important factors that should be considered if the factor of safety is to be reduced.

The load factor of 2.00 used in Chapters Four through Six made it probable that if prototype pile load tests were performed which included minimum strength piles (ϕ), the assumed eccentric loading, and normal pile damage (HDF), that a test to twice the working load could still be performed. The load factor of 2.00 can be reduced, but if done, there is a lower probability that a test pile can be loaded to twice the design load. Also, as discussed previously, a' test successfully completed to twice design loading is not proof in itself that ϕ is higher than 0.85, or that the eccentricity factor is inoperative.

For purposes of discussion, the higher allowable stresses derived for H-pile sections based on reduced load factors will be tabulated and examined.

H-PILES

Load Factor	fa as percent of Fy	Remarks
2.0	30	Recommended in this report
1.735	34	AASHTO Average Load Factor
1.7	35	Current Building Codes
1.55	38	PCA (Buildings)
1.4	42	ACI Dead Load Factor
1.3	46	Low Dead Load Factor
1.26	47	AASHTO Steel Column Design
1.19	50	AISI Recommendation

The reasons for recommending a load factor of 2.00 herein have been described previously, A factor of 1.735 is obtained for the average of AASHTO dead and live plus impact factors. On this basis an allowable of 0.34 Fy is obtained for use in bridges. Current building codes now allow 0.35 Fy which is consistent with a load factor of 1.7, For concrete piles, PCA recommended an average of the ACI dead and live load factors, 0.5(1.4 + 1.7), which is 1.55. If applied to H-piles, this results in an allowable stress of 0.38 Fy. Similarly, the ACI dead load factor of 1.4 results in an allowable of 0.42 Fy. Some codes based on ultimate strength design and partial factors of safety deal with load factors as low as 1.3; this results in an allowable stress of 0.46 Fy. At the extreme is the recommendation of the American Iron and Steel Institute, 0.5 Fy, which back-calculates to a load factor of 1.19. Finally, AASHTO design rules for short steel columns result in an allowable stress of 0.47 Fy for which a load factor of 1.26 is back-calculated.

Some perspective can be gained by the foregoing comparisons. There is no logic in allowing stresses in a steel pile equal or higher than that in a column. A column can be examined for damage, corrosion and deterioration, and also is maintainable; a pile lacks these advantages. Therefore, design pile stresses that approach the allowable stress used for steel columns seems overly bold. On the other hand, there is a considerable body of successful experience using design stresses of 0.35 Fy (where Fy = 36 ksi). Thus, the step from a load factor of 2.0 to 1.7 should not be too difficult. However, load factors of less than 1.7 put the design into the range where pile axial stiffness may be too little to allow driving the pile against the soil sufficiently hard to develop the desired pile-soil load capacity.

With respect to a procedure for higher allowable stresses, the following is suggested for steel piles:

1. Select the load factor and grade of steel to be employed.

2. Solve for fa in the governing equation, assuming $\phi = 0.85$, and ecc = 0.7 for H-piles and 0.91 for pipe piles.

3. If the resulting design stress exceeds 12,500 psi, a load test is required.

4. Where a pile is test loaded, use the maximum load obtained or the load at failure, whichever is less, divided by the cross-sectional area of the pile, to obtain the ultimate stress. In no event shall the ultimate stress be considered to exceed Fy for the pile material.

5. Insert the stress from step 4 above in place of Fy in the governing equation. Solve for the design stress assuming the values of ϕ and ecc used in step 2 above.

6. Modify the foregoing steps as required to account for differences between load transfer in the load test and that occuring during the service life of the structure.

It should be noted that the foregoing procedure assumes that ϕ = 1 and ecc =0 in the pile load test. There is no way of proving otherwise; thus, this procedure is conservative,

Concrete Piles - The design expressions for concrete piles given in this report are in terms of allowable load (Pa), not allowable stress. They were, however, derived using expressions similar to those for steel, with the form:

$$Pa = \phi\ (ecc)\ (HDF)\ (Po)/LF$$

Expressions have been given in this report for the nominal ultimate load, Po, for each type of concrete pile. A discussion analogous to that for steel is applicable, covering.,+, ecc and HDF. The designer always has the option of increasing the allowable load by increasing Po, which can be done by using higher strength concrete (f 'c) and/or reinforcing steel (fy). For a given pile material, however, an increase in allowable stress reduces the factor of safety. The allowable loads in Chapter Five were derived based on a load factor of 2.00, as for steel.

A procedure for achieving higher allowable loads can be listed in a manner analogous to that for steel, as follows;

1. Select the load factor and grades of concrete and reinforcing steel to be used.

2. Solve for Pa in the governing equation assuming the ϕ and ecc values used in Chapter Five for the type of pile involved.

3. If the resulting design stress (Pa/Ac) for precast or prestressed piles exceeds 1,600 psi, then a load test is required. If a top driven steel pipe pile is used and the design load divided by the pipe cross-sectional area exceeds 12,500 psi, then a pile load test is required.

4. Where a pile is test loaded, use the maximum load obtained or the load at failure, whichever is less, as Po. In no event, however, shall Po be considered greater than that calculated using the nominal strengths assumed in step 1 above.

5. Insert the Po value from step 4 above into the governing equation. Solve for the design load assuming the values of ϕ and ecc used in step 2 above.

6. Modify the foregoing steps as required to account for differences between load transfer in the load test and that occurring during the service life of the structure.

As with steel, the procedure assumes ϕ = 1 and ecc = 0 in the pile load test. There is no way of proving otherwise; therefore, this conservatism is warranted. .

For the purposes of developing perspective, the following load factors will be discussed:

Load Factor	Remarks
2.0	Recommended in this report
1.735	AASHTO Average Dead and Live
1.7	ACI Live Load Factor
1.55	PCA (Buildings)
1.4	ACI Dead Load Factor

The factors of 1.4 on dead load and 1.7 on live load are ACI standards for buildings. PCA used an average load *factor* of 1.55 *for* developing their • llwabla stress recommendations for concrete piles for buildings. It is considered reasonable to be more conservative with a bridge than with a building; therefore, a load factor of 1.7 to 1.735 (the average AASHTO load factor) may not be too bold a step. Further, there must be some precedent for using a load factor of 1.55 on bridges because current AASHTO allowable stresses involve load factors in this range. If the designer feels comfortable with the particular foundation situation, the use of a load factor of 1.55 is consistent with most codes governing building construction.

Timber - Because timber is a natural material whose manufacture is not under the control of the engineer, variations in strength properties greatly exceed those for concrete and steel. Complications also arise because of strength losses due to preservative treatment, variation in strength along the pile, and strength reduction with duration of loading (creep). The factors in the expression for allowable stress will be discussed with a view to increasing allowable stress; the expression is:

$$s_a = \phi\gamma\psi(ecc)\,(HDF)\,(s'_c)\,(\beta)/(FS)$$

where:

ϕ = strength ratio *for* defects (knots, etc.)
γ = adjustment factor butt to tip
ψ = adjustment factor for preservative treatment
ecc = eccentricity factor
HDF = Hidden Defect Factor
s'_c = 5 percent exclusion, small clear strength
β = adjustment factor for load duration
FS = near minimum factor of safety (1.25)

It is difficult to separate the discussion of using a stronger timber versus using a lower factor of safety as a means of increasing allowable stress; therefore, the two topics will be merged.

The product of the factors ϕ, y, and ψ is analagous to ϕ for steel and concrete. For timber, ϕ is a ratio applied to clear wood strength that accounts for defects, primarily knots. A way of increasing the allowable stress is to specify, purchase, and control by inspection the range of defects that are accepted in the trees used for piles. On the other hand, γ adjusts outt stre'ngth to pile tip strength; this cannot be controlled.

Preservative treatment is necessary for foundation piles, but the process causes a strength reduction. This reduction is maximum for steam conditioned piles. Higher allowables are shown in this report for timber piles that are not steam conditioned, meaning that other processes result in a higher value for ψ. However, treatment penetration and retention should not be sacrificed in attempting to increase allowable stresses by modifying the treatment process.

Eccentricity and the HDF are not subject to reduction, as with steel and concrete. The adjustment for duration of load (β) is already **part** of **AASHTO** timber design procedures; therefore, no new increases in allowable stress can be generated because the procedure for so doing is already operative.

The governing equation is based on modifying the basic small clear green butt strength, s'_c, which is determined by the ASTM D2555 procedure, This strength is arrived at statistically and is the 5 **percent** exclusion strength. In other words, 5 percent of the samples are weaker and 95 percent are stronger. Further, this value may be the result of combining several species. For example, southern pine piles are supposedly from four species: longleaf, shortleaf, slash and loblolly. A procedure for **arriving** at a higher allowable stress could consider selecting trees from stronger species or deleting weaker species from a group, such as deleting loblolly from the southern pine group.

Finally, the near minimum factor of safety of 1.25 could be reduced in order to arrive at a higher allowable stress. If the factor of safety is eliminated the implication is that generally one pile in twenty would then fail if it is subjected to the loadings used in the design assumptions. This is unsatisfactory to structural and foundation engineers; therefore, encroachment upon a factor of safety that is already minimal cannot be supported as sound engineering judgement . Because of the large natural variability in timber strength, the design allowable stress is made safe for the weaker members, based on a statistical procedure. It follows then that perhaps half of the piles in a foundation may be stronger than average and have a higher factor of safety than deemed necessary. This is necessary for safety because the material has highly variable properties.

In summary, the procedures for developing higher allowable stresses for timber are limited, and cannot be outlined as completely as was done for steel and concrete. The following steps are possible, although they may or may not be practical:

1. Change the specifications governing supply of the piles, requiring fewer defects, particularly knots. The data in Chapter Six can be used to extract a higher value of ϕ consistent with the specification.

2. Use methods of drying and preservative treatment resulting in the highest value of ψ, the adjustment factor for treatment.

3. Choose a stronger species of timber, and provide knowledgeable inspection to insure that only that species is used.

4. Eliminate the weaker timber species from a group of species normally used for piles, and provide knowledgeable inspection to insure that only the desired species are used.

5. If a pile is test loaded, the differences between load transfer in the test and during the service life of the structure must be considered .

LOAD SHARING CONCEPT

Foundation engineers have often speculated about the concept of load sharing applied to pile foundations, just as it is applied to other structural uses, notably in timber joists and rafters. The concept is used to justify a bold design, or one involving high allowable stresses. To be effective there must be a member or mechanism that causes adjacent structural members to act in conjunction with the member under discussion that is weak or failing. An assumption in the concept is that the weak member is ductile and carries what load it can, and any excess load is transferred to adjacent stronger members. For a pile foundation, load transfer would be through the pile cap.

The foregoing concept obviously cannot be applied to groups containing one, two or three piles because failure of any one pile makes the group unstable. In fact, one pile failing in a group of four piles would lead to tilting of the pile cap and an unsatisfactory performance. Thus, the load sharing concept is not worthy of discussion until the group contains at least five piles.

There are very few full-scale tests on groups of piles, especially those involving more than four piles. However, a review of those tests invariably shows a mode of failure beginning with tilting of the cap. Tilting is a natural outgrowth of relative weakness or failure of a single pile, particularly if it is strategically located. It is well known (Whitaker, 1957) that exterior piles attract more than the average load in a group of piles, and the corner piles attract more load than other exterior piles. Hence, it is more likely that a corner pile will fail or otherwise prove unsatisfactory; it also is obvious that a tilt of the pile cap would result.

Another factor that should be considered is eccentric loading of a group of piles caused by piles being driven outside of their design location, but still within the normal location tolerance permitted in most specifications. For example , a concentrically loaded 4-pile cluster with center-center spacing of 3 ft, and a location tolerance of 3 in. results in a maximum individual pile load of 124 percent of the average load. This result was arrived at by considering out-of-location piles and using the method of calculation given by Peck et al (1974). Thus, there are two phenomena acting on any normal group of piles that cause the maximum pile load to be higher than the average pile load. Neither of these phenomena indicate support for the load sharing concept.

A brief review of the load sharing concept a8 used in timber construction provide8 some background information. A 15 percent increase in bending stress is typically allowed for repetitive-member uses such as joists, trusses, raf ters, studs, planks, decking or similar member8 that are spaced not more than 24 inches, are not less than 3 in number and are joined by floor, roof or other load distributing element8 adequate to support the design load. Therefore, the past history refer8 to bending, not compression, and is applied to minor structural element8 such a8 joints, rafters and studs. Current use of the load sharing concept in timber construction doe8 not reveal any analogies to column8 or piles. For that reason, as well a8 the foregoing discussion, the load sharing concept is not recommended for use in pile foundations.

EVALUATION OF PILE LOAD TESTS

The foregoing procedure8 for arriving at higher-than-recommended allowable stresses involve driving and load testing a pile. A pile load test can be dangerously misleading unless interpreted properly. The discussion in Chapter Three of factor8 that increase load or decrease resistance are important, particularly the discussion of negative skin friction and load transfer analysis.

An apparently successful load test can be obtained in soil condition8 where, on a long-term basis, a failure of the foundation will result. Thus, it is necessary to consider the nature of the load tranefer acting during the test, and by analysis determine how it will subsequently exist in the pile foundation. Alternatively, the test can be configured to simulate long-term conditions; however, this is a difficult task and is seldom performed. Load transfer analyses are most important for non-prismatic piles, that is, tapered, stepped, composite, etc., because they usually become structurally weaker near the tip.

CHAPTER NINE
SUMMARY AND CONCLUSIONS

SUMMARY

A comprehensive review has been made of the factors entering into the structural strength of piling subjected to both axial and bending loads. This is an obvious necessary step in a rational procedure for arriving at allowable stresses. Moment-thrust interaction diagrams were developed for all types of piles considered herein consisting of steel, concrete, timber and composite sections.

The nominal strength of a pile section (PO) is based on nominal yield or strength (F_y, $0.85f'_c$) multiplied by the area of the pile cross-section. Then, reduction factors are introduced that account for the possible difference between the strength of an entire pile versus that indicated by a sample (coupon, cylinder). The reduction factor (ϕ) is referred to as the product (pile)/sample ratio. A O-value of 0.85 was determined for both axial load and bending in steel. For concrete pile sections O-values ranged from 0.60 to 0.75 in compression. Timber, by comparison, is more complicated and involves four reduction factors for defects, treatment, duration of load and position in the tree. The reduction factors in general allow the reliable strength of the pile cross section to be expressed as ϕ PO. This is an important step because reliable strength is required for Load Factor/Resistance Factor Design, which is likely to be adopted for foundations in the future.

The ACI concept of requiring all columns to be designed for some minimum bending has been adopted herein. This has the beneficial effect of accounting for large differences in bending strength about two axes, such as for H-sections. Thus, an eccentricity factor (ecc) is introduced that accounts for section shape. A minimum eccentricity of five percent of the pile width was adopted.

Pile sections are known to be damaged to varying degrees by driving. An attempt *to* allow for the strength reducing effects of damage has been introduced herein for the first time; it is a factor, HDF, called the hidden defect factor. Values of 1.0, 0.85 and 0.70 have been suggested for ideal, normal, and severe conditions, respectively. With HDF it is possible to express the reliable structural strength of the pile in the ground as ϕ (ecc) (HDF) (PO). The next step is to arrive at an allowable pile load by introducing a margin of safety.

Current AASHTO standards, and the engineering profession in general, require that load test piles be loaded to twice the design load or allowable load. Therefore, the reliable load capacity in the ground should be at least twice the allowable load. A load factor (LF) of 2.00 is introduced into the expression for allowable load (P_{all}) applicable to steel and concrete piles.

$$P_{all} = \phi\,(ecc)\,(HDF)\,(Po)/LF$$

This expression has also been used for timber piles with three modifications: (1) PO represents the 5 percent exclusion value, (2) the load factor is taken as 1.25 in recognition of the fact that PO is less than the average value of strength.

The foregoing development of a rational method of determining the structural strength of piling permits all pile materials to be treated consistently. Further, the basic expressions from which allowable stresses may be derived is in a form compatible with the load and resistance factor design method which may be adopted for foundations in the future.

The above expression for allowable load can be used to assess the risk involved in using allowable stresses that are higher than those recommended herein. This involves changing the load factor to some value less than 2.00. Such a procedure has been developed in Chapter Eight. The factors to be considered in arriving at allowable stresses were covered in Chapter Three, and the subjects of environmental factors and pile damage are expanded upon in Chapter Seven.

Pile material suppliers are active in lobbying code writing bodies for increases in allowable stresses. The claims of the material suppliers have been evaluated in Chapters Pour through Six by comparison to the expression for allowable load developed herein. It was found that the material suppliers all ignored one or more of the factors in the expression for allowable load. Further, their claims for the values of other factors were often overstated.

RECOMMENDATIONS

The factors described above were used to develop allowable stresses for steel, concrete, timber and composite steel/concrete piles. Recommended changes to the piling section of the AASHTO (1977) bridge specification were developed; these are fully presented in Chapter Ten as a suggested revision to the specification. Allowable stresses recommended in this report for the various types of piles are summarized in Tables 40 through 43. These tables are recommended replacements to Table 1.4.4 in the 1977 AASHTO specification.

Allowable stresses published by a wide array of code bodies were reviewed in Chapter Two. It is noted that the normal (HDF = 0.85) allowable stress for ASTM A36 steel piles recommended herein equals that currently allowed by AASHTO (9,000 psi, 62 MPa). Under ideal conditions (HDF = 1) the recommended allowable stresses are higher than those now permit ted by AASHTO. However, any allowable stresses recommended herein that are beyond 12,500 psi (86.2 MPa) require substantiation by load test and evaluation by the engineer. Similar provisions were developed for steel pipe piles. The steel allowable stresses recommended herein for ASTM A36 and A252 Gr.2 steels are slightly less than the base values used in most building codes; however, for higher strength steels higher allowables are provided.

In the case of concrete piles, the allowable loads recommended herein are lower than those in most building codes primarily because a load factor of 2.00 was used in this report instead of the value of 1.55 recommended by the Portland Cement Association. AASHTO (1977) allows stresses equivalent to those in most building codes. Therefore, the recommendations of this report amount to a reduction in allowable stress (load) relative to current AASHTO allowables. The impact of adopting the recommendations given in this report, however, is expected to be minor because most highway piling would, if checked by these recommendations, be found adequate.

TABLE 40 - STEEL PILES -ALLOWABLE STRESSES

Type	HIDDEN DEFECT FACTOR, HDF Ideal - 1.0	Normal - 0.85	Severe - 0.70
Rolled Shapes(1)	0.30 Fy(2)	0.25 Fy(2)	0.20 Fy2
ASTM A36 Fy-36 ksi (248 MPa)	10.8 ksi (74.5 MPa)	9.0 ksi (62.1 MPa)	7.2 ksi (49.7 MPa)
ASTM A252, GR50 Fy=50 ksi (345 MPa)	15.0 ksi (2) (103 MPa)	12.5 ksi (86.2 MPa)	10.0 ksi (69 MPa)
Pipe - Unfilled	0.39 Fy	0.33 Fy	0.27 Fy
ASTM A252, GR2 Fy=35 ksi (241 MPa)	13.65 ksi(2) (94.1 MPa)	11.55 ksi (79.7 MPa)	9.45 ksi (62.2 MPa)
ASTM A252, GR3 Fy=45 ksi (310 MPa)	17.55 ksi(2) (121)MPa)	14.85(2)ksi (102 MPa)	12.15 ksi (83.8 MPa)

(1) Shapes must meet the requirements of 1.7.59(B), except that Mu shall be taken as 0.85 FyS. Shapes not satisfying Table 1.7.59B are subject to reduction to satisfy 1.7.5.9(B)(1)(a).

(2) Allowable stresses exceeding 12.5 ksi (86.2 MPa) may be used only where load tests and evaluation by the engineer confirm satisfactory results.

(3) For combined bending and axial load adapt 1.7.45 or 1.7.69 and modify as required to satisfy footnote #1 above. For strength design net interaction diagrams:
(1) rolled shapes; straight line from Po = 0.85 FyAs(HDF)(R) to Mo = 0.85 FyS(HDF)(R) where R is a reduction factor for non-compact sections not satisfying Table 1.7.59B; Pmax = 0.70 Po, and
(2) Pipe; curve $M/Mo = \cos(0.5\pi P/Po)$; PO = 0.85 FyAs(HDF); MO = 0.85 Fy(1.3S)(HDF); Pmax = 0.91 Po.

TABLE 41 - PRECAST PILES - ALLOWABLE STRESSES [1][2]

Type	HIDDEN DEFECT FACTOR, HDF		
	Ideal - 1.0	Normal - 0.85	Severe - 0.70
Precast with reinforcing bars	**0.26f'cAc+0.30fyAs**	**0.22f'cAc+0.26fyAs**	**0.18f'cAc+0.21fyAs**
Prestressed	**(0.26f'c-0.21fce)Ac**	**(0.22f'c-0.18fce)Ac**	**(0.18f'c-0.15fce)Ac**

(1) Minimum f'c to be 5000 psi (34.5 **MPa)**

(2) For combined bending and axial load adapt section 1.5.33 except that in **1.5.33(A)(2)(b,c)** the minimum eccentricity shall be **0.05w** where w is the width or diameter of the pile. The resulting nominal interaction diagram shall be reduced for a ϕ of 0.70 in compression and 0.90 in bending per 1.5.30(B); further reductions shall be made for **HDF.**

TABLE 42 - CAST-IN-PLACE PILES - ALLOWABLE STRESSES (1)(2)

Type	HIDDEN DEFECT FACTOR, HDF				
	Ideal - 1.0	Normal	HDF	Severe	HDF
Pipe:(5) ϕ=0.75 added reinforcement	0.28(f'cAc+FyAs) +0.33fyAsr+0.28FysAss	0.25(f'cAc+FyAs) +0.29fyAsr+0.25FysAss	0.90	Eliminate by on-site insp.	--
Shell:(Mandrel driven) '#=0.65 added reinforcement	0.25 f'cAc +0.29fyAsr+0.25FysAss	0.21 f'cAc +0.25fyAsr+0.21FysAss	0.85	Eliminate by on-site insp.	--
Shell w/confinement(3)(4) 4=0.70 added reinforcement	0.30 f'cAc +0.29fyAsr+0.25FysAss	0.25 f'cAc +0.25fyAsr+0.21FysAss	0.85	Eliminate by on-site insp.	--
Uncased ϕ=0.60 added reinforcement	0.23 f'cAc +0.27fyAsr+0.23FysAss	0.19 f'cAc +0.22fyAsr+0.19FysAss	0.85	0.15 f'cAc +0.18fyAsr +0.15FysAss	0.70

(1) Minimum f'c = 2500 psi (17.2 MPa)

(2) For combined bending and axial load adapt section 1.5.33 except that in 1.5.33(A)(2)(b,c) the minimum eccentricity shall be 0.05w where w is the width or diameter of the pile. The resulting, nominal interaction diagram shall be reduced by the $-factors given above for compression and 0.90 in bending per 1.5.30(B); further reductions shall be made for HDF.

(3) Applies only where: (1) corrosion will not occur, (2) f'c does not exceed 5000 psi (34.5 MPa), (3) diameter does not exceed 16 3/8 in. (416 mm), (4) shell thickness is not less than 0.075 in. (1.98 mm), and (5) the yield strength of the shell is 30 ksi (207 MPa) minimum as proven by tests on representative samples.-

(4) Nominal interaction diagram is determined by constructing diagram for unconfined concrete. Then determine Po' as 1.2Po, where Po is that for unconfined concrete. Draw line from Po' tangent to diagram for unconfined concrete.

(5) For top-driven pipe, a load test(s) evaluated by the engineer to confirm satisfactory results is required when the pile design load exceeds 12.5 ksi (86.2 MPa) on the cross-sectional area of pipe alone.

TABLE 43 - TIMBER PILES - ALLOWABLE STRESSES(1)(2)(6)

Species	Location	Pile Length	HIDDEN DEFECT FACTOR, HDF(7): Ideal, HDF=1.0 — Untreated (3)	Ideal, HDF=1.0 — Treated(4) Boulton Process	Ideal, HDF=1.0 — Treated(4) Steamed	Normal, HDF=0.85 — Untreated (3)	Normal, HDF=0.85 — Treated(4) Boulton Process	Normal, HDF=0.85 — Treated(4) Steamed
Douglas Fir	Butt	All	900	750	--	750	650	--
	Tip	50 ft. or less	800	700	--	700	600	--
	Tip	Over 50 ft.	700	600	--	600	525	--
Southern Pine	Butt	All	900	--	650	750	--	550
	Tip	50 ft. or less	800	--	600	700	--	580
	Tip	Over 50 ft.	700	--	550	600	--	450
Other(5)	Butt	All	$0.35s'_c$	$0.30s'_c$	$0.26s'_c$	$0.30s'_c$	$0.25s'_c$	$0.22s'_c$
	Tip	50 ft. or less	$0.31s'_c$	$0.27s'_c$	$0.24s'_c$	$0.27s'_c$	$0.23s'_c$	$0.20s'_c$
	Tip	Over 50 ft.	$0.29s'_c$	$0.24s'_c$	$0.22s'_c$	$0.24s'_c$	$0.21s'_c$	$0.18s'_c$

Notes

(1) Assumes piles are furnished according to quality standard AASHTO M168.

(2) Allowable stresses are for normal duration; for other loading conditions, adjustment should be made as given in Sections 1.10.1 C, D and E.

(3) Subject to the use limitations of Section 1.4.5-B.

(4) Treatment conditioning (Boulton or Steaming) shall be selected as the most severe permitted under AASHTO M 133.

(5) s'_c is the 5% exclusion limit in compression parallel to grain for small, green, clear wood specimens per ASTM D 2555-78.

(6) For combined axial and bending stresses, attention is directed to Section **1.10.1F**; however, in no case shall the design eccentricity be less than 0.05 D where D is the pile diameter.

(7) Timber piles are not recommended for severe conditions, unless field driving and extraction test verifies that piles can be installed without structural damage or damage to the treatment to safely carry the required load.

Timber pile allowable stresses recommended in this report are significantly below those apparently allowed by AASHTO (1977). This arose because timber piling had not been treated by a rational analysis until the year 1979. The saving feature has been that timber piles were generally not used for loads beyond 24 tons (213.5 kN) for piles with a 12-inch (305 mm) butt because they tend to break easily during driving. This effectively kept actual pile stresses in the range of those recommended here in. Thus, it is believed that if AASHTO adopted the allowable stresses recommended in this report it would have very little effect on the current utilization of timber piles.

Recommendations have also been given for driving stress limitations. A limit of 1.1 fy is recommended for steel, and 0.85 - 1.0 f'c for concrete. A firm recommendation for timber is difficult to develop; a value of 3.0 times the recommended allowable static design stress is suggested.

CHAPTER TEN
IMPLEMENTATION OF RECOMMENDED ALLOWABLE STRESSES

INTRODUCTION.

The allowable pile stresses derived in this report are applicable to pile foundations for highway bridges and other related highway structures. Implementation of this research requires that the research results' be converted to language suitable for specifications. The information developed has been incorporated into suggested revisions to 'the 1977 MSHTO Standard Specifications for Highway Bridges. The suggested revisions are included herein.

Specifically, Section 1.4.1 through 1.4.6 of the 1977 AASHTO Code have been reviewed and revisions are suggested that are pertinent to pile design; in addition, several small changes are suggested that are primarily clarifications. Some changes are also suggested for section 2.3.13 on the manufacture of precast concrete piles to bring it into conformity with current design/construction practice.

The following pages first discuss the basic expression for deriving allowable pile stresses, then, the changes and the reasons for the changes are listed, and finally, applicable MSHTO specifications are modified. Revisions through the 1983 Interim Specifications have been included.

PILE ALLOWABLE STRESSES

A unified procedure was used for assessing the strength of piles as structural columns. Because lateral support from the soil surrounding a pile prevents buckling of the embedded portions of the pile it is possible to treat piles as short columns. This simplifies the assessment of strength. Moment-thrust interaction diagrams were developed for each type of pile based on nominal strengths and dimensions of the materials involved. Then strength reduction factors were applied to the nominal strength to arrive at net pile strength. A factor of safety was applied to net strength to arrive at the working load or allowable stress.

Some minimum bending was considered in arriving at allowable axial stresses. This was done by introducing an eccentricity of 0.05 times the width of the pile. Eccentricity factors were determined from the moment-thrust diagrams for each type of pile.

Strength reduction factors or e-factors were considered for each type of pile. The factors considered included understrength in the basic materials as finally configured after placement in the pile.

Pile strength reduction caused by damage due to driving the pile into the ground is introduced under the heading of "hidden defect factor" (HDF). Three conditions are defined, as follows:

Ideal - Soil conditions are known not to cause damage to the type of pile selected.

Normal - Soil conditions are not expected to cause damage to the type of pile selected.

Severe - Soil conditions are known to cause significant damage to the type of pile selected.

For ideal conditions HDF is unity, For solid pile cross-sections and normal sites the HDF is 0.85; for severe site conditions the HDF is 0.70. For pipe and normal site conditions the HDF is 0.9.

The factor of safety has been set at 2.0 on the reduced strength of the in-place pile. Two is considered the smallest factor that can be used because of pile load test requirements to two times the design load.

Allowable stresses can be determined from the resulting expression for allowable load:

$$P_{all} = \frac{(F)(A)(ecc)(\phi)(HDF)}{LF}$$

where: F = Nominal strength of the material (F_y, 0.85 $f'c$, etc.)
A = Cross-sectional area of the pile
ecc= Eccentricity factor
ϕ = Strength reduction factor
HDF= Hidden Defect Factor
LF = Load factor or safety factor (2.0)

Where axial load and other than minimum bending exists it is generally desirable to resort to interaction diagrams for design. This is easily accomplished by factoring the loads from the strength design provisions of AASHTO and comparing them to the net strength from the interaction diagram. (Net as used here means after reduction for eccentricity, strength reduction ϕ, and hidden defects HDF.) Alternatively, the designer could use working stress design and take the allowable as some percentage the strength determined from the net interaction diagram.

The factors necessary to construct moment-thrust interaction diagrams are given in the tables and/or in the footnotes to the tables in the proposed revision of AASHTO Section 1.4.4 (B) Case A, Capacity of Pile as a Structural Member, for structural members not commonly treated in reference texts. Thus, the proposed revision provides a set of rules from which pile strength in situ can be determined, and compared to factored loads from the MSTHO strength design provisions. When AASHTO develops a strength design specification for foundations, the proposed revisions, if adopted, will be compatible.

The foregoing discussion applies directly to concrete and steel piles. A generally similar approach was applied to timber piles, but the strength reduction factors, ϕ, are more complicated. Also, timber suffers from long term effects or creep. Essentially, under a given load the factor of safety of a timber member decreases with time. The proposed revision allows for such behavior in a manner similar to that of Section 10, Timber Structures, of the current AASHTO specification.

It will be noted that the proposed revision generally lowers the allowable stresses for concrete and timber piles, but increases them for steel piles. The reductions for concrete and timber relate to the introduction of strength reduction factors, ϕ, and the hidden defect factor, HDF. The eccentricity or minimum moment factor also contributed to the decrease. It is believed that the proposed revision represents a consistent and rational set of allowable stresses that considers the appropriate factors in assessing structural strength.

RECOMMENDED CHANGES

MSHTO PAGE NO.	REVISION NO.	REASON FOR CHANGE
50	1	Clarifies possible conflict if Section 4 covers a subject that is also covered in Sections 5, 6, 7 and 10. Thus, Section 4 would control.
50	2	Change footnote reference to the current edition of Terzaghi & Peck.
52	1	This is the major change recommended herein. The full explanation of how the tables were arrived at is covered in the FHWA report titled, **Allowable Stresses in Piles". A summary is given herein in the following section.
53	1	Replaced with revision of page 52.
54	1	Replaced with revision of page 52.
55	1	The table applies where subsurface investigation or test loads have not been performed. Therefore, it is prudent to assume severe subsoil conditions for which an allowable stress of 7200 psi was derived.
56	1	Existing paragraph may be unconservative for high capacity piles in that it suggests 40 percent of the compression capacity as the tension capacity, without limit on the load. The suggested replacement allows for determination of tension capacity by load test or calculation, and also accounts for the difference between a single pile and a group of piles.

AASHTO PAGE NO.	REVISION NO.	REASON FOR CHANGE
56	2	The existing Converse-Labarre formula has been shown by M.F. Ghanem ("Bearing Capacity of Friction Piles in Deep Soft Clays," Ph.D. Thesis, University of Illinois, 1953) to be an attempt on the basis of elastic theory to achieve uniform pile settlement. The soil mechanics techniques in the suggested revision are thought to be superior in that they address both bearing capacity and settlement. Further, the Converse-Labarre formula has almost universally been rejected in modern design practice.
58	1,2	The existing size limitations date back to the year 1927 when concrete strengths were lower and pile sixes larger to carry a given load. The suggested revision allows a 10-in. square pile to be used, similar to that now allowed in prestressed piles. In salt water a 12-in. square pile can be used, also similar to that now allowed for prestressed piles. This change would bring the AASHTO Code into conformity with current good engineering practice for precast piles.
58	3	It is not considered good engineering practice to have a pile tip size of less than 8 in, because of potential damage during driving. The suggested change also clarifies where tip size is measured.
58		The suggested change still requires a minimum *of* 1.5 percent steel and 4 bars; however, if more than 4 bars are used it is not left to the designer to configure the steel to meet the 1.5 percent requirement and to satisfy bar development lengths required elsewhere by AASHTO concrete design rules. It is felt that reinforced concrete design techniques in Section 5 cover bar discontinuity and make the current statement under the heading of precast piles unnecessary.

MSHTO PAGE NO.	REVISION NO.	REASON FOR CHANGE
59	1	The suggested revision clarifies the meanings of shell (usually corrugated) and pipe. Also, the minimum thicknesses of each material are specified and the requirement for no corrosion is added in the event thicknesses less than 1/16 in. are used for structural purposes. (Note that Section 1.4.5 (K), page 60, mentions deducting 1/16 in. for corrosion.)
62		The suggested revision correlates design with the construction provision, Section 2.3.4 (Ii) on page 296, that allows piles to be driven up to 6 inches out-of-position. This has an important impact on pile cap design and can make the difference between no shear on a section and an important shear on the section; a pile cap failure could result under the current provisions, but is prevented under the suggested revision.
63A	1	A recent project in which pile caps failed due to shear has prompted some forthcoming changes in the ACI Code warning of shear in deep beams. This suggested revision provides proper warning and provides references on the subject.
64	1	The suggested revision is that this statement be deleted because it is theoretically incorrect.
64	2	It is not recommended that pile caps be unreinforced.
299	1	The current wording dates back to the year 1927 when precast concrete practice was much different. Such wording makes current precast plant practices' economically unusable. The current provisions are justifiably not met by any precast suppliers because other provisions of most codes including AASHTO provide sufficient safeguards.
300	1	The suggested revision adds a statement clarifying current good practice.

RECOMMENDED CHANGES
TO
SECTIONS 1.4 & 2.3 OF THE 1977 MSHTO
STANDARD SPECIFICATION FOR HIGHWAY BRIDGES*

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(1) Section 1.4.1(B), line 3 - After "section 10" change period to comma, and add "unless otherwise limited herein."

(2) Section 1.4.2, footnote - Revise reference to "2nd Edition of 1967".

Pare 52

(1) Section 1.4.4(B)(1) - Delete both paragraphs and replace with the following:

The embedded portions of piles shall generally be designed as structural columns according to the allowable loads and stresses given in the following tables:

Steel Piles - Table 1.4.4A
Precast and Prestressed Piles - Table 1.4.4B
Cast-In-Place Piles - Table 1.4.4C
Timber Piles - Table 1.4.4D

The designer shall select a hidden defect factor (HDF) appropriate to the soil conditions expected at the job site, as follows:

Ideal - Soil conditions are known not to cause damage to the type of pile selected.
Normal - Soil conditions are not expected to cause damage to the type of pile selected.
Severe - Soil conditions are known to cause significant damage to the type of pile selected.

Free-standing portions of piles down to the point at which the soil provides lateral support, and design details not covered in tables 1.4.4A through 1.4.4D shall be according to the section of this specification governing the pile material involved, as follows:

Steel - Section 7
Concrete - Section 5
Prestressed Concrete - Section 6
Timber - Section 10

TABLE 1.4.4A - STEEL PILES - ALLOWABLE STRESSES

Type	HIDDEN DEFECT FACTOR. HDF Ideal - 1.0	Normal - 0.85	Severe - 0.70
Rolled Shapes[1]	0.30 Fy[2]	0.25 Fy[2]	0.20 Fy[2]
ASTM A36 Fy=36 ksi (248 MPa)	10.8 ksi (74.5 MPa)	9.0 ksi (62.1 MPa)	7.2 ksi (49.7 MPa)
ASTM A252, GR50 Fy=50 ksi (345 MPa)	15.0 ksi [2] (103 MPa)	12.5 ksi (86.2 MPa)	10.0 ksi (69 MPa)
Pipe - Unfilled	0.39 Fy	0.33 Fy	0.27 Fy
ASTM A252, GR2 Fy=35 ksi (241 MPa)	13.65 ksi[2] (94.1 MPa)	11.55 ksi (79.7 MPa)	9.45 ksi (62.2 MPa)
ASTM A252, GR3 Fy=45 ksi (310 MPa)	17.55 ksi[2] (121) MPa)	14.85[2] ksi (102 MPa)	12.15 ksi (83.8 MPa)

(1) Shapes must meet the requirements of 1.7.59(B), except that Mu shall be taken as 0.85 FyS. Shapes not satisfying Table 1.7.59B are subject to reduction to satisfy 1.7.5.9(B)(1)(a).

(2) Allowable stresses exceeding 12.5 ksi (86.2 MPa) may be used only where load tests and evaluation by the engineer confirm satisfactory results.

(3) For combined bending and axial load adapt 1.7.45 or 1.7.69 and modify as required to satisfy footnote #1 above. For strength design net interaction diagrams:
(1) rolled shapes; straight line from $P_o = 0.85\ F_yA_s(HDF)(R)$ to $M_o = 0.85\ F_yS(HDF)(R)$ where R is a reduction factor for non-compact sections not satisfying Table 1.7.59B; $P_{max} = 0.70\ P_o$, and
(2) Pipe; curve $M/M_o = \cos(0.5\pi P/P_o)$; $P_o = 0.85\ F_yA_s(HDF)$; $H_o = 0.85\ F_y(1.3S)(HDF)$; $P_{max} = 0.91\ P_o$.

TABLE 1.4.4B - PRECAST PILES - ALLOWABLE LOADS(1,2)

Type	HIDDEN DEFECT FACTOR. HDF		
	Ideal = 1.0	Normal = 0.85	Severe = 0.70
Precast with reinforcing bars	$0.26f'_cA_c+0.30f_yA_s$	$0.22f'_cA_c+0.26f_yA_s$	$0.18f'_cA_c+0.21f_yA_s$
Presttessed	$(0.26f'_c-0.21f_{ce})A_c$	$(0.22f'_c-0.18f_{ce})A_c$	$(0.18f'_c-0.15f_{ce})A_c$

(1) **Minimum f'c** to be 5000 psi (34.5 **MPa**)

(2) For **combined bending** and axial load adapt section 1.5.33 except that in **1.5.33(A)(2)(b,c)** the minimum eccentricity shall be **0.05w** where w is the width or diameter of the pile. The resulting nominal interaction diagram shall be reduced for a ϕ of 0.70 in compression and 0.90 in bending per 1.5.30(B); further reductions shall be made for **HDF.**

TABLE 1.4.4C - CAST-IN-PLACE PILES - ALLOWABLE LOADS(1)(2)

Type		HIDDEN DEFECT FACTOR, HDF			
	Ideal = 1.0	Normal	HDF	Severe	HDF
Pipe:(5) ϕ=0.75 added reinforcement	0.28(f'cAc+FyAs) +0.33fyAsr+0.28FysAss	0.25(f'cAc+FyAs) +0.29fyAsr+0.25FysAss	0.90	Eliminate by on-site insp.	--
Shell: (Mandrel driven) ϕ=0.65 added reinforcement	0.25 f'cAc +0.29fyAsr+0.25FysAss	0.21 f'cAc +0.25fyAsr+0.21FysAss	0.85	Eliminate by on-site insp.	--
Shell w/confinement(3)(4) ϕ=0.70 added reinforcement	0.30 f'cAc +0.29fyAsr+0.25FysAss	0.25 f'cAc +0.25fyAsr+0.21FysAss	0.85	Eliminate by on-site insp.	--
Uncased ϕ=0.60 added reinforcement	0.23 f'cAc +0.27fyAsr+0.23FysAss	0.19 f'cAc +0.22fyAsr+0.19FysAss	0.85	0.15 f'cAc +0.18fyAsr +0.15FysAss	0.70

(1) Minimum f'c = 2500 psi (17.2 MPa)

(2) For combined bending and axial load adapt section 1.5.33 except that in 1.5.33(A)(2)(b,c) the minimum eccentricity shall be 0.05w where w is the width or diameter of the pile. The resulting, nominal interaction diagram shall be reduced by the &factors given above for compression and 0.90 in bending per 1.5.30(B); further reductions shall be made for HDF.

(3) Applies only where: (1) corrosion will not occur, (2) f'c does not exceed 5000 psi (34.5 MPa), (3) diameter does not exceed 16 3/8 in. (416 mm), (4) shell thickness is not less than 0.075 in. (1.98 mm), and (5) the yield strength of the shell is 30 ksi (207 MPa) minimum as proven by tests on representative samples.

(4) Nominal interaction diagram is determined by constructing diagram for unconfined concrete. Then determine Po' as 1.2Po, where Po is that for unconfined concrete. Draw line from Po' tangent to diagram for unconfined concrete.

(5) For top-driven pipe, a load test(s) evaluated by the engineer to confirm satisfactory results is required when the pile design load exceeds 12.5 ksi (86.2 MPa) on the cross-sectional area of pipe alone.

TABLE 1.4.4D - TIMBER PILES - ALLOWABLE STRESSES(1)(2)(6)

Species	Location	Pile Length	Ideal. HDF=1.0 Untreated (3)	Ideal. HDF=1.0 Treated(4) Boulton Process	Ideal. HDF=1.0 Treated(4) Steamed	Normal. HDF=0.85 Untreated (3)	Normal. HDF=0.85 Treated(4) Boulton Process	Normal. HDF=0.85 Treated(4) Steamed
			HIDDEN DEFECT FACTOR, HDF(7)					
Douglas Fir	Butt	All	900	750	--	750	650	--
	Tip	50 ft. or less	800	700	--	700	600	--
	Tip	Over 50 ft.	700	600	--	600	525	--
Southern Pine	Butt	All	900	--	650	750	--	550
	Tip	50 ft. or less	800	--	600	700	--	500
	Tip	Over 50 ft.	700	--	550	600	--	450
Other(5)	Butt	All	0.35s'c	0.30s'c	0.26s'c	0.30s'c	0.25s'c	0.22s'c
	Tip	50 ft. or less	0.31s'c	0.27s'c	0.24s'c	0.27s'c	0.23s'c	0.20s'c
	Tip	Over 50 ft.	0.29s'c	0.24s'c	0.22s'c	0.24s'c	0.21s'c	0.18s'c

Notes

(1) Assumes piles are furnished according to quality standard AASHTO M168.

(2) Allowable stresses are for normal duration; for other loading conditions, adjustment should be made as given in Sections 1.10.1 C, D and E.

(3) Subject to the use limitations of Section 1.4.5-B.

(4) Treatment conditioning (Boulton or Steaming) shall be selected as the most severe permitted under AASHTO M 133.

(5) s'_c is the 5% exclusion limit in compression parallel to grain for small, green, clear wood specimens per ASTM D 2555-78.

(6) For combined axial and bending stresses, attention is directed to Section 1.10.1F; however, in no case shall the design eccentricity be less than 0.05 D where D is the pile diameter.

(7) Timber piles are not recommended for severe conditions, unless field driving and extraction test verifies that piles can be installed-without structural damage or damage to the treatment to safely carry the required load.

Page 53

(1) Delete all items on this page.

Page 54

(1) Delete paragraph "(e)" at top of page.

Page 55

(1) In the table at the bottom of the page under the column labeled **Steel Point-Bearing** change 9000 to 7200 and 62.05 to 49.7.

Page 56

(1) Section 1.4.4(F)(1) - Replace entire paragraph with the following:

The uplift design capacity for a single pile shall not exceed 33 percent of the ultimate frictional capacity determined by a static analysis method. Alternatively, the uplift capacity of a single pile can be determined by load tests according to ASTM D-3689. If determined by load tests the allowable uplift design capacity shall be not greater than 50 percent of the failure uplift load.

The allowable working uplift load for a pile group shall be the lesser of: (1) The individual pile design uplift load multiplied by the number of piles in the group, or (2) 2/3 of the effective weight of the pile group and the soil contained within a block defined by the perimeter of the group and the length of the piles, or (3) 1/2 the effective weight of the pile group and soil contained within a block defined by the perimeter of the group and the pile length plus 1/2 the total shear on the peripheral surface of the group.

(2) Section 1.4.4(G) - Replace entire paragraph with the following:

With respect to compressive loads, it is not necessary to consider group efficiency except for a group of friction piles in cohesive soil. In cohesive soils the design load on a group of friction piles shall not exceed 50 percent of the capacity determined from an ultimate load analysis involving the ultimate bearing capacity of the soils within the plan area of the group plus the shearing resistance of the soil on the perimeter of the group. Also, the design load on the group shall not exceed 50 percent of the ultimate capacity of each individual pile in the group multiplied by the number of piles in the group. The settlement of the pile group shall not exceed the tolerable settlement limits of the structure.

Page 58

(1) First paragraph, lines 4 and 5 - Change 140 to 98, and ($0.0903m^2$) to ($0.0632m^2$).

(2) First paragraph, line 6 - Change 220 to 140, and ($.1419m^2$) to ($.0903m^2$).

(3) Second paragraph - Delete and replace with the following:

The diameter of tapered piles measured at the point shall be not less than 8 inches (.203 m). In all cases the diameter shall be considered as the least dimension through the center,

(4) Third paragraph - Delete and replace with the following:

Vertical reinforcement shall be provided consisting of not less than four bars spaced uniformly around the perimeter of the pile. It shall be at least 1-1/2 percent of the total cross section measured above the taper.

Page 59

(1) Second paragraph, line 8 - After the words Where the shell is" delete remainder of the paragraph and replace with:

smooth pipe and is more than 0.12 inch (3 mm) in thickness, it may be considered as load carrying in the absence of corrosion. Where the shell is corrugated and is at least 0.075 inch (1.89mm) in thickness, it may be considered as providing confinement in the absence of corrosion.

Page 62

(1) Section 1.4.6(E)(1) - At end of the paragraph add the following sentence:

Piles shall be considered displaced horizontally up to 6 inches (150mm) from their theoretical location so as to produce the most critical design condition.

Pane 63A

(1) At the conclusion of Section 1.4.6(H) add the following:

(d) Footings shall be analyzed as deep flexural members, where applicable; for one way shear, see ACI 318-77 Section 11.8; for two way (slab) shear, see CRSI Handbook, 1980, Section 13.

Page 64

(1)(2) Delete the last sentence of the first paragraph.

Pane 299

(1) Section 23.13(C) - Retain the first two sentences in the paragraph and delete the remainder.

Page 300

(1) Section 2.3.13(G) - Add the following sentence to the end of the paragraph:

Concrete need not be protected from freezing beyond the time the compressive strength reaches the smaller of 0.8f'c or 4000 psi (27.6 MPa).

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