U.S. Department
of Transportation

**Federal Highway
Administration**

Guide Design Specification for Bridge Temporary Works

FOREWORD

This study is part of the Federal Highway Administration's Temporary Works Research Program, conducted as a result of the falsework collapse of the Route 198 bridge over the Baltimore/Washington Parkway, Maryland. This report is intended to serve as a guide for designing temporary works for highway bridge structures. The report addresses falsework, formwork, and temporary retaining structures. Although intended for use by the falsework design engineer, this report will also be of interest to inspection engineers and contractors.

Additional copies may be obtained from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

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Acting Director, Office of
Engineering and Highway
Operations Research
and Development

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16. Abstract

Following the collapse of the Route 198 bridge over the Baltimore/Washington Parkway in 1989, the FHWA established the temporary works research program. The program was guided by the Scaffolding, Shoring, and Forming Task Group as formed by the FHWA.

The objective of this study has been to develop a guide design specification for use by State agencies to update their existing standard specifications for falsework, formwork, and related temporary construction. The guide specification was prepared in a format similar to the AASHTO Standard Specifications for Highway Bridge Structures.

This report is one in a series of reports produced under this program which include:

FHWA REPORT NO.

RD-91-062	Synthesis of Falsework, Formwork, and Scaffolding for Highway Bridge Structures
RD-93-031	Guide Standard Specification for Bridge Temporary Works
RD-93-033	Certification Program for Bridge Temporary Works
RD-93-034	Construction Handbook for Bridge Temporary Works

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PREFACE

Objective

In 1991, a study was initiated by the Federal Highway Administration (FHWA) to identify the current state of the practice in the United States and abroad for designing, constructing, and inspecting the falsework and formwork used to construct highway bridge structures. The findings of this study were published in FHWA-RD-91-062, *Synthesis of Falsework, Formwork, and Scaffolding for Highway Bridge Structures.*[1]

As part of the aforementioned study, a questionnaire was developed and sent to the fifty U.S. highway departments. Information relating to design and administrative policies for falsework and formwork construction, and the bridge construction activity for each State was requested. Virtually every State was found to have general requirements and guidelines for the construction and removal of falsework and formwork. However, only about half of the States specified design criteria. Similarly, only 22 States had accompanying design or construction manuals that included specific design information.

In addition to identifying the content of State specifications, the survey also provided some insight regarding each State's respective administrative policies concerning falsework and formwork. About two-thirds of the States require the submittal of plans and calculations, sealed by a registered professional engineer, for any significant falsework construction. By definition, significant falsework generally corresponds to anything that spans over 16 ft (4.9 m) or rises more than 14 ft (4.3 m) in height. The survey showed that a majority of States also conduct their own reviews and inspections, subject to availability of staff, complexity of design, and so forth. While the proposed design specification herein does not include procedural guidelines, recommended guidelines can be found in the *Guide Standard Specification for Bridge Temporary Works* recently issued by the FHWA.[2]

Based upon a review of current State practice, the authors of the Synthesis concluded that design procedures vary considerably from one State to another. As expected, States that are more active constructing cast-in-place concrete highway bridges generally have more comprehensive specifications and design guidelines. There are also many States, however, with significant bridge inventories that do not fall into this group. Therefore, despite the available information, there appeared to be a clear need to develop unified design criteria and standards for the temporary structures used to construct highway bridges.

The objective of this study has been to develop a guide design specification for use by State agencies to update their existing standard specifications for falsework, formwork, and related temporary construction. The guide specification was prepared in a format similar to the American Association of State Highway and Transportation Officials (AASHTO) *Standard Specifications for Highway Bridge Structures.*[3]

Research Approach

Virtually all of the potential reference or source documents necessary to develop the guide design specification and commentary were identified in the Synthesis noted above. The reference material included applicable AASHTO standards; the standard specifications of the FHWA and several States; Canadian, British, and New Zealand codes of standard practice and specifications; and standards or guides prepared by other agencies and industry associations.[3-17] While the existing foreign standards served as good models, they were not found to be entirely adaptable to U.S. codes and construction practices or the proprietary systems common to the U.S. construction industry. Where applicable, selected provisions of other standards were either adopted directly from the reference material or slightly modified. However, no single reference document was found that covered the entire scope of the guide design specification.

After reviewing the reference material, a preliminary outline was developed by Wiss, Janney, Elstner Associates, Inc. (WJE) and submitted to an industry advisory group for further comments. The outline was, in turn, submitted to the FHWA for approval. The preliminary draft was developed in the same manner, then revised after receiving the collective comments of the advisory group and the FHWA.

Throughout the study, the primary objective was to develop a consensus document that would be readily adopted by the State agencies and AASHTO. The evaluation criteria in the project guidelines stipulated that the proposed guide specification reflect generally accepted practice, be acceptable to industry, and be supported by existing research. The final draft was revised to reflect the consensus of WJE and the advisory group, the FHWA, and other individuals or associations identified in the acknowledgement. In addition, a commentary was developed to provide background on the provisions and to assist in the application of this specification.

Recommendation for Implementation

The following guide design specification closely reflects the current state of the practice for the falsework, formwork, and temporary retaining structures used in highway bridge construction and is offered for adoption by AASHTO.

The authors recommend distributing this document to the appropriate State agencies for use in revising their current standard specifications. The authors also recommend soliciting further comments from the State bridge authorities, AASHTO, and industry groups. After trial use of this specification, it is recommended that the FHWA Task Group reconvene to consider any revisions or additions that may be proposed.

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in²	square inches	645.2	square millimeters	mm²
ft²	square feet	0.093	square meters	m²
yd²	square yards	0.836	square meters	m²
ac	acres	0.405	hectares	ha
mi²	square miles	2.59	square kilometers	km²
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft³	cubic feet	0.028	cubic meters	m³
yd³	cubic yards	0.765	cubic meters	m³

NOTE: Volumes greater than 1000 l shall be shown in m³.

Symbol	When You Know	Multiply By	To Find	Symbol
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m²	cd/m²
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in²	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm²	square millimeters	0.0016	square inches	in²
m²	square meters	10.764	square feet	ft²
m²	square meters	1.195	square yards	yd²
ha	hectares	2.47	acres	ac
km²	square kilometers	0.386	square miles	mi²
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m³	cubic meters	35.71	cubic feet	ft³
m³	cubic meters	1.307	cubic yards	yd³
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m²	candela/m²	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in²

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

(Revised September 1993)

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ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
AISC	American Institute of Steel Construction
AISI	American Iron and Steel Institute
AITC	American Institute of Timber Construction
ANSI	American National Standards Institute
APA	American Plywood Association
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
AWS	American Welding Society
BOCA	Building Officials & Code Administrators
FHWA	Federal Highway Administration
NAVFAC	Naval Facilities Engineering Command
NDS	National Design Specification for Wood Construction
NFPA	National Forest Products Association
OSHA	Occupational Safety and Health Administration
PCI	Precast/Prestressed Concrete Institute
SSFI	Scaffolding, Shoring and Forming Institute
UBC	Uniform Building Code

GENERAL NOTATION

in.	inches
ft	feet
plf	pounds per linear foot
psi	pounds per square inch
ksi	kips per square inch
psf	pounds per square foot
ksf	kips per square foot
tsf	tons per square foot
pcf	pounds per cubic foot
fps	feet per second
m	meters
N	newtons
hr	hour

INTRODUCTION

1.1 SCOPE

This guide design specification has been developed for use by State agencies to include in their existing standard specifications for falsework, formwork, and related temporary construction used to construct highway bridge structures. The specification should also be useful to bridge engineers, falsework designers, contractors, and inspectors. Sections within this specification address falsework, formwork, and temporary retaining structures. Reference standards, related publications, and definitions are identified below.

1.2 REFERENCES

1.2.1 Codes and Standards

Standard Specifications for Highway Bridges, Latest Edition, American Association of State Highway and Transportation Officials, Washington, DC.

Interim Specifications Bridges, 1991, American Association of State Highway and Transportation Officials, Washington, DC, 1991.

Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects (FP-85), Federal Highway Administration, Washington, DC, 1991.

American National Standard for Construction and Demolition Operations: Concrete and Masonry Work-Safety Requirements (ANSI A10.9-1983), American National Standards Institute, New York, NY, 1982.

American National Standard for Construction and Demolition Operations: Safety and Health Program Requirements for Multi-Employer Projects (ANSI 10.33-1992), American National Standards Institute, New York, NY, 1992.

Manual of Steel Construction-Allowable Stress Design, Ninth Edition, American Institute of Steel Construction, Chicago, IL, 1989.

Structural Welding Code-Steel (AWS D1.1), American Welding Society, Miami, FL, 1988.

National Design Specification for Wood Construction, 1991 Edition, National Forest Products Association, Washington, DC, 1991.

NDS Supplement-Design Values for Wood Construction, National Forest Products Association, Washington, DC, 1991.

Building Code Requirements for Reinforced Concrete (ACI 318-89) and Commentary (ACI 318R-89), American Concrete Institute, Detroit, MI, 1989.

Minimum Design Loads for Buildings and Other Structures (ASCE 7-88), American Society of Civil Engineers, New York, NY, 1990.

Uniform Building Code, 1991 Edition, International Conference of Building Officials, Whittier, CA, 1991.

The BOCA National Building Code/1990, Building Officials & Code Administrators International, Inc., Country Club Hills, IL, 1990.

Occupational Safety and Health Standards (Parts 1910 and 1926), United States Department of Labor, Occupational Safety and Health Administration, Washington, DC, 1974.

1.2.2 Related Publications

Guide Standard Specification for Bridge Temporary Works (FHWA-RD-93-031), Federal Highway Administration, Washington, DC, 1993.

Certification Program for Bridge Temporary Works (FHWA-RD-93-033), Federal Highway Administration, Washington, DC, 1993.

Construction Handbook for Bridge Temporary Works (FHWA-RD-93-034), Federal Highway Administration, Washington, DC, 1993.

California Falsework Manual, California Department of Transportation, Sacramento, CA, 1988.

Guide to Formwork for Concrete (ACI 347R-88), American Concrete Institute, Detroit, MI, 1989.

Formwork for Concrete (SP-4), Fifth Edition, American Concrete Institute, Detroit, MI, 1989.

Timber Construction Manual, Third Edition, John Wiley & Sons, Inc., New York, NY, 1985.

Guidelines for Safety Requirements for Shoring Concrete Formwork, Publication No. SH306, Scaffolding, Shoring, and Forming Institute, Inc., Cleveland, OH, 1990.

Recommended Procedure for Compression Testing of Welded Frame Scaffolds and Shoring Equipment, Publication No. S102, Scaffolding, Shoring, and Forming Institute, Inc., Cleveland, OH, 1989.

Analysis and Design of Reinforced Concrete Bridge Structures (ACI 343R-88), American Concrete Institute, Detroit, MI, 1988.

Soil Mechanics, Foundations, and Earth Structures (NAVFAC DM-7), Department of the Navy, Alexandria, VA, May 1982.

Suggested Design and Construction Procedures for Pier Foundations (ACI 336.3 R-72), American Concrete Institute, Detroit, MI, Revised 1985.

1.3   DEFINITIONS

For the purposes of this specification, the following definitions apply:

Brace - A member placed diagonally with respect to the vertical or horizontal members of falsework or scaffolding and fixed to them to provide stability.

Cofferdam - A cofferdam is a watertight structure that allows foundations to be constructed under dry conditions.

Engineer - This term used with a capital "E" refers to the owner's engineer.

Factor of Safety - The ratio of predicted ultimate load to the calculated maximum service load.

Falsework - Temporary construction used to support the permanent structure until it becomes self-supporting. Falsework would include steel or timber beams, girders, columns, piles and foundations, and any proprietary equipment including modular shoring frames, post shores, and horizontal shoring.

Formwork - A temporary structure or mold used to retain the plastic or fluid concrete in its designated shape until it hardens. Formwork must have enough strength to resist the fluid pressure exerted by plastic concrete and any additional fluid pressure effects generated by vibration.

Horizontal Shoring Beams - Adjustable or fixed length beams or trusses used as load-carrying members in falsework systems.

Allowable Stress - The stress that can be sustained with acceptable safety by a structural component under the particular condition of service and loading.

Post Shore - Individual vertical member used to support loads, including adjustable timber single-post shores, fabricated single-post shores, and timber single-post shores.

Scaffolding - An elevated work platform used to support workmen, materials, and equipment, but not intended to support the structure being constructed.

Shoring - A component of falsework such as horizontal, vertical, or inclined support members. For the purpose of this document this term is used interchangeably with falsework.

Temporary Retaining Structure - For the purpose of this document, temporary retaining structure refers to both earth-retaining structures and cofferdams.

Tube and Coupler Shoring - An assembly used as a load-carrying structure, consisting of tubing or pipe that serves as posts, braces and ties, a base supporting the posts, and special couplers that serve to connect the uprights and join the various members.

Ultimate Load - The maximum load that may be placed on a structure, causing failure by buckling of column members or failure of some other component.

1.4   METRIC CONVERSIONS

Conversion equations from U.S. Customary units to S.I. metric units are provided in Appendix E.

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Section 2

FALSEWORK

2.0 FALSEWORK DRAWINGS

All elements of the falsework system shall be shown on working drawings, hereinafter referred to as falsework drawings. The falsework drawings shall include the information and details necessary to enable the falsework to be constructed without reference to any supplemental drawing, calculation sheet, design standard, or other source or reference document.

The falsework drawings shall include all design-controlling dimensions, including beam length, beam spacing, post location and spacing, vertical distance between connections in diagonal bracing, height of falsework bents, and similar dimensions controlling falsework design and erection.

The falsework drawings shall include a superstructure placing diagram, which shall show the concrete placing sequence, placement rate, and all construction joint locations.

When footing-type foundations are to be used, the soil-bearing value assumed in the design shall be shown on the falsework drawings.

When pile-type foundations are to be used, and the vertical distance between the ground line and the top of the pile will equal or exceed four times the pile diameter at the ground line, the falsework drawings shall show the maximum horizontal distance that the top of a falsework pile may be pulled to its position under the cap, and the maximum allowable deviation of the top of the pile, in its final position, from a vertical line through the point of fixity of the pile.

The anticipated total settlement of the falsework and forms shall be shown on the falsework drawings. The anticipated settlement shall include both foundation settlement and joint take-up, and shall not exceed 1 in. (25 mm).

The falsework drawings shall show the method by which the falsework may be adjusted vertically, and the locations where such adjustments will be reinforced.

Where openings through the falsework are required to permit the passage of public traffic, including pedestrian traffic, the falsework drawings shall show the location of all such openings, including horizontal and vertical clearances and the location of temporary railing.

Where temporary bracing is to be used during erection and removal of falsework over or adjacent to public traffic, the falsework drawings shall show the sequence of erection and removal and details of the temporary bracing system to be used.

The falsework drawings, when submitted to the owner, shall be accompanied by one set of the design calculations. The calculations shall show the stresses and deflections in load-supporting members.

2.1 MATERIALS AND MANUFACTURED COMPONENTS

2.1.1 General

Falsework design may be based on the use of either new or used materials and manufactured components, or a combination thereof.

2.1.2 Structural Steel

2.1.2.1 Identification and Properties

New structural steel shall conform to the ASTM and AASHTO specifications designated in Table 2.1. The most recent date of issue shall apply to each of these specifications.

Certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6 or A568, as applicable, and the governing specification shall constitute sufficient evidence of conformity with one of the above ASTM standards. Additionally, the fabricator shall, if requested, provide an affidavit stating that the structural steel furnished meets the requirements of the grade specified.

For new structural steel, the design working stresses shall not exceed those specified in the *AISC Specification for Structural Steel Buildings-Allowable Stress Design.* For structural steel design, the modulus of elasticity (E) shall be assumed as 29,000 ksi (200,000 N/mm^2).

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Table 2.1 Material Properties

Type	Structural Steel	Welded and Seamless Steel Pipe	Carbon Steel Structural Tubing			Carbon Steel
			A500 Round	A500 Square and Rectangular	A501	A529
Equivalent ASTM Designation	A36 Grade 36	A53 Grade B	A500 Round	A500 Square and Rectangular	A501	A529
AASHTO Designation	M183 Grade 36	---	---	---	---	---
Minimum Tensile Strength (F_u), psi	58,000	60,000	Grade A: 45,000 Grade B: 58,000 Grade C: 62,000	Grade A: 45,000 Grade B: 58,000 Grade C: 62,000	58,000	60,000
Minimum Yield Strength (F_y), psi	36,000	35,000	Grade A: 33,000 Grade B: 42,000 Grade C: 46,000	Grade A: 39,000 Grade B: 46,000 Grade C: 50,000	36,000	42,000

Conversion: 1,000 psi = 6.89 N/mm^2

2.1.2.2 Salvaged Steel

Used structural steel, satisfying ASTM A6 criteria for surface imperfections, may be used at the allowable working stresses for new material, provided the grade of steel can be identified to the owner's satisfaction.

When the grade of used structural steel cannot be identified, the design working stresses shall not exceed the following:

Tension, axial, and flexural . . . . 22,000 psi

Compression, axial . . . . . . 16,000 - 0.38(L/r)^2 psi, except L/r shall not exceed 120.

Shear on gross section of web of rolled shapes . . . . 14,500 psi

Web crippling for rolled shapes . . . . . . 16,000 psi

Compression, flexural . . . . 12,000,000/(Ld/bt) psi, but not more than 22,000 psi.

In the formulas, L is the unsupported length; d is the least dimension of rectangular columns, or the width of a square of equivalent cross-sectional area for round columns, or the depth of beams; b is the width and t is the thickness of the compression flange; and r is the radius of gyration of the member.

2.1.2.3 Welding

All provisions of the *Structural Welding Code, AWS D1.1-92*, of the American Welding

Society, except 2.3.2.4, 2.5, 8.13.1.2, and Section 9, as appropriate, apply to work performed under this specification.

2.1.3 Timber

2.1.3.1 Allowable Stresses

All species of wood to which allowable unit stresses have been assigned in the *National Design Specification for Wood Construction (NDS) Supplement*, 1991 edition, are acceptable for use in falsework.

Design working stresses for new lumber shall not exceed the design values for visually graded dimension lumber and visually graded timbers as tabulated in the *National Design Specification for Wood Construction (NDS) Supplement*, 1991 Edition. The listed values are for normal load duration and dry service conditions, and shall be modified as provided herein.

2.1.3.2 Modification Factors

Modification factors for service conditions and duration of load shall be as prescribed by NDS except that the normal service condition for falsework members shall be considered to be dry and reduction for wet service conditions will not apply. All modification factors are cumulative.

Load duration factors shall not apply to values for modulus of elasticity or compression perpendicular to the grain.

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Table 2.1 Material Properties (Cont.)

Type	High Yield Strength, Quenched and Tempered Alloy Steel	High-Strength Low-Alloy Steel		High-Strength Low-Alloy Structural Tubing
Equivalent ASTM Designation	A514	A572	A588	A618
AASHTO Designation	M244	M223	M222	---
Minimum Tensile Strength (F_u), psi	Grade 100 up to 2½ in.: 110,000 Grade 100 over 2½ in. to 6 in.: 100,000	Grade 42: 60,000 Grade 50: 65,000	70,000	Grades I & II: 70,000 Grade III: 65,000
Minimum Yield Strength (F_y), psi	Grade 100 up to 2½ in.: 100,000 Grade 100 over 2½ in. to 6 in.: 90,000	Grade 42: 42,000 Grade 50: 50,000	50,000	Grades I, II, & III: 50,000

Conversion: 1,000 psi = 6.89 N/mm²; 1 in. = 25.4 mm

2.1.3.3 Used Lumber

Subject to the owner's concurrence, used lumber of known species may be used in accordance with the following:

(a) Where the grade is known or can be established, the stress level for used lumber, in good condition and without obvious defects, shall not exceed the adjusted allowable stress for new lumber of that grade and species.

(b) Where the grade is unknown and cannot be established, the stress level for used lumber, in good condition and without obvious defects, shall not exceed the adjusted allowable stress for No. 1 commercial grade new lumber of that species.

(c) The stress level for used lumber of lower quality or showing evidence of abuse shall not exceed the stress values for the species as listed in Appendix A. The listed stress values are maximums and shall not be increased by application of load duration or other stress-adjustment factors.

Unless otherwise permitted by the owner, lumber of unknown species may not be used at higher stress levels than those listed in Appendix A for "MIXED MAPLE." Said stresses are maximums and shall not be increased by application of load duration or other stress-adjustment factors.

The owner may require any lumber proposed for use under paragraphs (a) or (b) above to be regraded prior to use.

2.1.4 Other Materials

The design of materials other than structural steel and timber shall conform to the applicable design standard or specification for such material.

2.1.5 Manufactured Components

2.1.5.1 General

As used herein, manufactured components include the following classes of proprietary products:

- Vertical shoring systems including tubular welded frame shoring, tube and coupler shoring, and components thereof.

- Manufactured assemblies including single-post shores, brackets, jacks, joists, clamps, and similar devices manufactured for commercial use.

2.1.5.2 Maximum Loadings and Deflections

The maximum load to be used on any manufactured component, under any load sequence or combination, shall not exceed the manufacturer's recommendations.

When requested by the owner, a manufacturer's catalog, technical bulletin, or similar publication shall be furnished with the falsework drawings showing the use of manufactured components. The information furnished shall include,

but not be limited to, test data and limitations and conditions governing the use of the component.

The dead load deflection of a manufactured component designed for use in a horizontal or inclined position shall not exceed 1/240 of the span length under the weight of the concrete only.

The use of a manufactured assembly for which no engineering data is furnished will not be permitted, unless the assembly has been tested under conditions simulating the proposed use in the falsework design. The working load for such assemblies shall not exceed 40 percent of the maximum load sustained during the test.

2.1.5.3 Factor of Safety

The factor of safety for vertical shoring systems shall not be less than 2.5. This shall be clearly evident from a catalog or other engineering data furnished by the manufacturer.

The factor of safety for jacks that are not a part of a shoring system, and all types of manufactured assemblies, shall not be less than the minimum factor of safety required by the industry standard for the particular device, and in no case shall the factor of safety be less than 2.

2.2 LOADS

2.2.1 General

The falsework design load shall consist of the sum of the dead and live vertical loads and a horizontal load.

The vertical design load shall consist of the sum of the dead and live vertical loads, including live load impact where appropriate.

The horizontal design load shall consist of the sum of any actual horizontal loads due to equipment, construction sequence, or other causes, excluding the specified wind load, but in no case shall the horizontal design load be less than 2 percent of the total dead load to be supported at the point under consideration.

Pursuant to the provisions in Section 2.3.1, the vertical and horizontal design loads shall be increased as necessary to account for the effect of load redistribution due to prestressing, shrinkage, or other causes.

If the effect of a particular loading condition cannot be determined at the falsework design stage, the design shall be based on an assumed loading condition. In such cases, the assumptions shall be reviewed when the actual conditions become known, and the falsework design revised if necessary.

2.2.2 Dead Load

The dead load shall consist of the weight of the concrete, construction materials, and falsework to be supported at any given location.

The combined weight of concrete, reinforcing and prestressing steel, and formwork shall be assumed to be not less than 160 pcf (25.1 kN/m^3) for normal concrete or 130 pcf (20.4 kN/m^3) for lightweight concrete.

2.2.3 Live Load

2.2.3.1 Construction Live Load

The construction live load shall consist of the actual weight of any equipment to be supported applied as concentrated loads at the points of contact, plus a uniform load of 20 psf (960 N/m^2) applied over the area supported, plus 75 plf (1100 N/m) applied at the outside edge of deck overhangs.

2.2.3.2 Impact

When impact can occur, the design load to be applied to steel members and manufactured components shall be increased as provided herein.

- For members and components subject to impact during placing operations, the design dead load shall be increased by an impact factor of not less than 30 percent of the weight of the material being placed.

- For members and components subject to impact during lifting operations, the static load due to the payload shall be increased by not less than 30 percent for mechanically operated lifting equipment and not less than 15 percent for manually operated lifting equipment.

- If motorized carts are used, the uniform live load shall be increased an additional 25 psf (1200 N/m^2).

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2.2.4 Minimum Vertical Load

The minimum total design vertical load for any falsework member shall be not less than 100 psf (4800 N/m²) for the combined dead and live load, exclusive of any increase for impact, regardless of slab thickness.

2.2.5 Environmental Loads

2.2.5.1 Wind

For heavy duty shoring systems having a vertical load-carrying capacity exceeding 30 kips (130 N) per tower leg, the minimum horizontal load to be allowed for wind shall be determined in accordance with Chapter 23, Part II of the Uniform Building Code (reproduced in Appendix C). The wind impact area shall be the total projected area of all elements in the tower face normal to the applied wind. The basic wind pressure for each height zone shall be increased by 5 psf (240 N/m²) for falsework members over or adjacent to traffic openings.

The minimum horizontal load to be allowed for wind on all other types of falsework, including falsework supported on heavy duty steel shoring, shall be the sum of the products of the wind impact area and the applicable wind pressure value for each height zone listed in Table 2.2. The basic wind speed used in the determination of design wind loads shall be as given in Fig. 2.1. The wind impact area shall be the gross projected area of the falsework and any unrestrained portion of the permanent structure, excluding the areas between falsework posts or towers

where diagonal bracing is not used. The basic wind pressure for each height zone shall be increased by 5 psf (240 N/m²) for falsework members over or adjacent to traffic openings.

2.2.5.2 Stream Flow

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

$$P_w = Kv^2 \qquad (2-1)$$

In the formula, P_w is the pressure in psf; v is the water velocity in fps; and K is a constant that shall take the following values:

1.375 for square faces
0.67 for circular piers
0.5 for angular faces

Where a significant amount of drift lodged against a pier is anticipated, the effects of this drift build-up shall be considered in the design. When it is anticipated that the flow area will be significantly blocked by drift build-up, increases in high water elevations, stream velocities, stream flow pressures, and the potential increases in scour depths shall be investigated.

2.2.5.3 Snow

Where necessary, the effects of snow shall be considered, and determined in accordance with ASCE 7-88 (formerly ANSI A58.1).

Table 2.2 Wind Pressure Values (P = $C_e C_q q_s I)^{(a)}$

Height Zone (ft above ground)	Pressure, psf for Indicated Wind Velocity, mph			
	70	80	90	100
0 to 30	1.5 Q	2.0 Q	2.5 Q	3.0 Q
30 to 50	2.0 Q	2.5 Q	3.0 Q	3.5 Q
50 to 100	2.5 Q	3.0 Q	3.5 Q	4.0 Q
over 100	3.0 Q	3.5 Q	4.0 Q	4.5 Q

Notes:
(a)   Refer to Appendix C for specific information on variables. When using U.S. Customary Units, the value of Q in Table 2.2 shall be determined as follows: Q = 1 + 0.2 W, but not more than 10. In the preceding formula, W is the width of the falsework system, in ft, measured in the direction of the wind force being considered.
(b)   Conversion: 1 ft = 0.3048 m; 1 psf = 47.88 N/m²; 1 mph = 1.609 km/hr
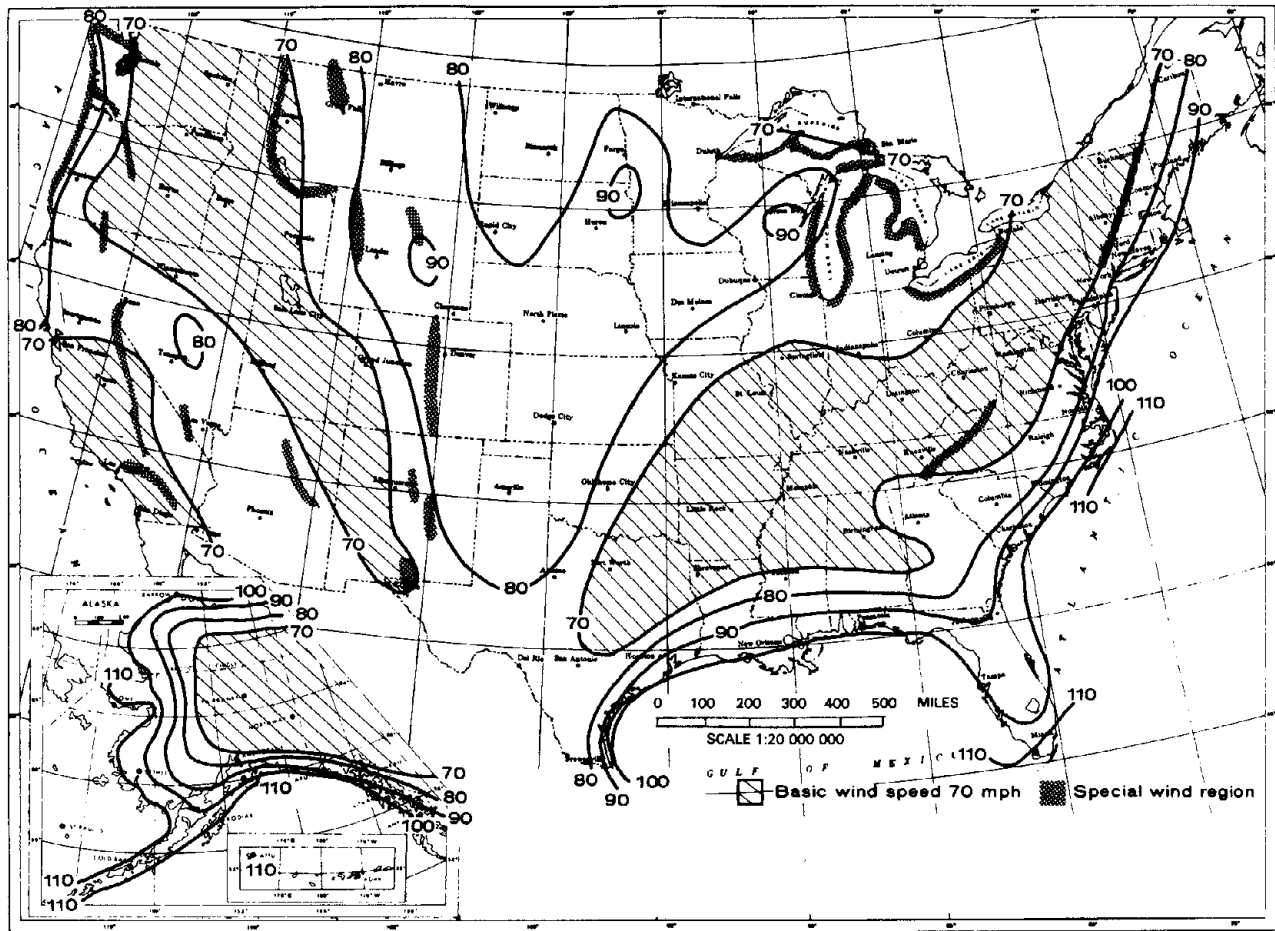
Figure 2.1 Basic Wind Speed (mph)

Notes:
(a) Values are fastest-mile speeds at 33 ft (10 m) above ground for exposure category C and are associated with an annual probability of 0.02.
(b) Linear interpolation between wind speed contours is acceptable.
(c) Caution in the use of wind speed contours in mountainous regions of Alaska is advised.
(d) Conversion: 1 mph = 1.609 km/hr; 1 mi = 1.61 km

2.3   DESIGN

2.3.1   General

The falsework design analysis shall consider the effect of foundation settlement, interaction between elements of the falsework system and completed portions of the permanent structure, and load redistribution due to shrinkage and dead load deflection. The falsework design shall accommodate these factors if necessary. For cast-in-place prestressed construction, the falsework shall be designed to support any increased load resulting from load redistribution caused by the prestressing forces.

The entire superstructure cross section, except railing, shall be considered to be placed at one time except as otherwise provided herein. Girder stems and connected bottom slabs, if placed more than 5 days prior to the top slab, may be considered to be self-supporting between falsework posts at the time the top slab is placed, provided that the distance between falsework posts does not exceed four times the depth of the portion of the girder placed in the first pour.

The support system for form panels supporting concrete deck slabs and overhangs on girder bridges shall be considered to be falsework and

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shall meet all falsework design criteria and requirements. Additionally, such falsework shall be designed so no differential settlement will occur between the girders and the deck forms during placement of the deck concrete.

2.3.2 Load Combinations

The groups given in Table 2.3 represent combinations of loads and forces to which the falsework may be subjected.

All elements of the falsework, or the foundation upon which it rests, shall be designed to resist the Group I, II, III, and IV load combinations at the percentage of the basic allowable stress shown.

2.3.3 Stability Against Overturning

The falsework system, including individual elements and units of the system that are subject to overturning forces, shall be analyzed for stability against overturning with the falsework in the loaded and unloaded condition; that is, with and without the dead load of the concrete. The ratio of the resisting moment to the overturning moment shall be equal to or greater than 1.2 for all load combinations.

Except as otherwise provided in the following paragraph, if the ratio of the resisting to the overturning moments is less than 1.2, external bracing shall be provided to resist the full overturning moment.

Except for bracing required to prevent overturning or collapse of the falsework system or any element of the system, the ability of falsework members to resist horizontal loads may include the contribution to stability provided by the supported structure. Bracing required to prevent overturning or collapse shall be designed to resist the full horizontal design load with the falsework in the unloaded (before the concrete is placed) condition, except for cable bracing used to externally brace heavy-duty shoring systems. Such cables may be designed to resist the difference between the overturning and resisting moments.

2.3.4 Combined Stresses

The adequacy of falsework members subjected to both axial and bending stresses shall be determined by the following combined stress expression:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0 \qquad (2-2)$$

where, respectively, f_a and f_b are the calculated axial and bending stresses and F_a and F_b are the allowable axial and bending stresses.

2.3.5 Deflection

The calculated vertical deflection for falsework members shall not exceed 1/240 of their span under the dead load of the concrete only, regardless of the fact that deflection may be compensated for by camber strips.

Table 2.3 Load Combinations

Group	Load Combinations	Percentage of Basic Allowable Stress or Load
Group I	DL + DP + LL + I + H	100%
Group II	DL + DP + PS + H	100%
Group III	DL + DP + LL + I + W + ALL	133%
Group IV	DL + DP + LL + PS + W + ALL	133%

where DL = design dead load;
DP = dead load of supported permanent structure;
LL = construction live load;
I = impact load;

H = minimum horizontal design load;
PS = redistributed prestress load;
W = wind load;
ALL = all other loads, including stream flow and snow.

2.3.6 Slenderness

For compression members, the slenderness ratio, Kl/r, shall not exceed the following:

(a)   Main load-carrying members:

    (i)   Steel - 180;
    (ii)   Aluminum - 100.

(b)   Bracing members:

    (i)   Steel - 200;
    (ii)   Aluminum - 150.

The slenderness ratio of a tension member, other than guy lines, cables, and rods, shall not exceed 240 for a main member nor 300 for a bracing member. These limits may be waived if other means are provided to control flexibility, sag, vibration, and slack in a manner commensurate with the service conditions of the structure, or if it can be shown that such factors are not detrimental to the performance of the structure or of the assembly of which the member is a part.

2.3.7 Steel Beam Grillages

Webs and flanges of steel beams under concentrated loads shall satisfy the criteria specified in Chapter K, *AISC Specification for Structural Steel Buildings - Allowable Stress Design*, reproduced in Appendix B.

2.3.8 Proprietary Shoring Systems

Differential leg loading of vertical shoring systems shall be minimized. In cases where differential leg loading cannot be avoided, the manufacturer of the shoring system shall furnish a letter of certification stating that the proposed loading differential will not overstress any tower component.

2.3.9 Traffic Openings

The vertical loads used for the design of falsework columns and towers, but not footings, which support the portion of the falsework over or immediately adjacent to open public roads, shall be increased to not less than 150 percent of the design loads that would otherwise be calculated in accordance with these provisions.

Each column or tower frame supporting falsework over or immediately adjacent to an open public road shall be mechanically connected to its supporting footing at its base, or otherwise be laterally restrained, so as to withstand a force of not less than 2,000 lb (8,900 N) applied to the base in any direction. Such columns or frames shall also be mechanically connected to the falsework cap or stringer so as to be capable of withstanding a horizontal force of not less than 1,000 lb (4,450 N) in any direction.

Temporary bracing shall be provided, as necessary, to withstand all imposed loads during erection, construction, and removal of any falsework whose height exceeds its clear distance to either the edge of any sidewalk or shoulder of any roadway that is open to the public or to a point 10 ft (3 m) from the centerline of any railroad track. The falsework drawings shall show such temporary bracing or methods to be used to conform to this requirement during each phase of erection and removal. Wind loads shall be included in the design of such bracing or methods.

2.4   FOUNDATIONS

2.4.1 General

The falsework shall be founded on pads, footings, or piles that are capable of carrying the imposed loads without aggravated distortion or settlement until the supported structure becomes self-supporting.

2.4.2 Footings

Footings shall be designed to distribute the applied load over the supporting foundation material uniformly, without exceeding the allowable soil-bearing value. The allowable soil-bearing value shall be determined by the Contractor through an examination of the site, a geotechnical foundation investigation, or through other appropriate means. In the absence of other soils information, the presumptive bearing values shown in Table 2.4 may be used as a guide, but the Contractor is responsible for the actual value used in the design.

The presumptive bearing values shown in Table 2.4 are for level and sloping ground where the slope is not greater than 1 vertical to 6 horizontal.

If the data obtained during the site exploration is minimal, or if there is a significant amount of variation from one sampling location to another, the presumptive bearing pressure value given in Table 2.4 shall be multiplied by a factor of 0.75 to

take into account the non-uniformity and uncertainty of the site conditions.

These values apply for the ground water level at a depth below the foundation greater than the width of that foundation. Continued flooding or wet weather will soften clay soils. Where site flooding and/or high ground water levels are likely to be experienced, the presumed allowable bearing pressure in Table 2.4 shall be multiplied by the factor given in Table 2.5.

Pursuant to the provisions in Section 2.0, the allowable soil-bearing value used in the foundation design shall be shown on the falsework drawings.

Footings with eccentric loadings shall be designed so that no portion of the footing has uplift pressures. Footings with lateral loading shall be designed to provide a 1.5 factor of safety against sliding.

2.4.3 Pile Foundations

When timber piles are used, the load applied to any pile in the foundation under any loading condition shall not exceed 45 tons (400 kN).

When steel piles are used, the load applied to any pile in the foundation under any loading condition shall not exceed the capacity of the pile when analyzed as a short column.

When piles extend above the ground line, the load-carrying capacity of both individual piles and piles within a framed bent shall be evaluated under the combined action of the vertical and horizontal design loads.

2.4.4 Foundations for Heavy-Duty Shoring Systems

Foundations for individual steel towers where the maximum leg load exceeds 30 kips (130 kN) shall be designed and constructed to provide uniform settlement under all legs of each tower under all loading conditions.

2.5 CONSTRUCTION

2.5.1 General

The falsework shall be constructed to conform to the falsework drawings. The materials used in the falsework construction shall be of the quality necessary to sustain the stresses required by the falsework design. The workmanship shall be of such quality that the falsework will support the loads imposed on it without excessive settlement or take-up beyond that shown on the falsework drawings.

2.5.2 Foundations

Falsework footings shall bear uniformly on the supporting material, which shall be safe from undermining and protected against softening.

When requested by the owner, the Contractor shall demonstrate by suitable load tests that the soil-bearing values assumed for the falsework design do not exceed the supporting capacity of the foundation material.

The load-carrying capacity of driven piles, unless driven by a drop hammer, shall be determined by the ENR or other recognized pile driving formula. If a drop hammer is used, the allowable pile capacity shall not exceed the value indicated by the driving formula divided by a factor of safety of 1.5.

2.5.3 Timber Construction

Timber beams, stringers, and joists shall be of the size and timber grade specified, and shall be straight and undamaged.

Adequate splice details shall be used where the splicing of timber members is unavoidable. Wherever practical, continuous or overlapping members shall be used.

Where the use of discontinuous members is permitted, joints shall be made over the center of supports. Where a beam comprises a pair of members, joints in the members shall be staggered between supports. Paired members used to form a single beam shall be of identical depth.

2.5.4 Steel Construction

Adequate bracing shall be provided to sections of members, in compression, and to resist lateral forces applied to the falsework.

Used beams, and particularly beams salvaged from a previous commercial use, shall be carefully examined for loss of section due to welding, rivet or bolt holes, or web openings that may adversely affect the beam's ability to safely carry the imposed load.

Table 2.4 Presumptive Soil-Bearing Values

Group	Foundation Bearing Deposit		Presumptive Bearing Pressure
1 Excellent Foundation Support	A.	Hard shales and soft sandstones	20 tsf
	B.	Soft shales, soft claystones, and very soft sandstones	6 to 10 tsf
	C.	Weak and fractured limestone	6 tsf
	D.	Dense sands and gravels	4 tsf
	E.	Very stiff to hard clays	3 to 4 tsf
2 Generally Adequate Foundation Support	A.	Medium dense sands and gravels	2 to 4 tsf
	B.	Medium dense uniform-size sand	2 tsf
	C.	Stiff clays	1 to 2.5 tsf
3 Poor Foundation Support	A.	Loose sand or loose sand and gravel	1 tsf
	B.	Loose uniform-size sand	0.75 tsf
	C.	Soft to medium clays	0.5 tsf
	D.	Loose silts	0.5 tsf
4 Unacceptable	A.	Peat and organic silts	Requires deep foundations for soils A and B
	B.	Very soft clays	
5 Potential Problem Soils	A.	Collapsible soils	Requires special attention to moisture control for soils A and B Could require insulation Could be same as groups 2 and 3 depending upon compaction
	B.	Swelling soils	
	C.	Frost-susceptible soils	
	D.	Fill deposits	

Conversion: 1 tsf = 96 kN/m²

Table 2.5 Ground Water-Level Modification Factors

Condition	Modification factors for:		
	Cohesive soils	Noncohesive soils	Rock
Ground water level at B, or less, below level of foundation (where B is the width of foundation)	1.0	0.5	1.0
Site liable to flooding	0.67	0.5	1.0

2.5.5 Proprietary Shoring Systems

Proprietary shoring systems shall be undamaged and assembled using only the components supplied by the manufacturer for the particular system.

Proprietary systems shall be installed in accordance with the manufacturer's recommendations, with provision for vertical adjustment.

Extension tubes shall be braced as required and erection tolerances shall not exceed the tolerances recommended by the manufacturer.

2.5.6 Manufactured Components

When manufactured components are used in the falsework, the owner shall be furnished with a letter of certification signed by the supplier of the component or his authorized representative. Said letter of certification shall state that the component is being used in accordance with the manufacturer's recommendations for loads and conditions of use.

2.5.7 Noncommercial Components

If the falsework construction incorporates generic or homemade components fabricated from steel or timber, such as overhang brackets, beam supports and similar devices, the owner may require a load test to establish the safe load-carrying capacity of such a component. Such tests shall be performed in the field, on components randomly selected by the owner, under conditions that will simulate the intended use in the falsework. The allowable capacity of any such component shall not exceed 40 percent of the ultimate load-carrying capacity as indicated by the load test.

2.5.8 Traffic Openings

Unless otherwise provided, the minimum dimensions of clear openings to be provided through falsework for roadways that are to remain open to traffic during construction shall be at least 5 ft (1.5 m) greater than the width of the approach traveled way, measured between barriers when used, and 14 ft (4.3 m) high, except that the minimum vertical clearance over interstate routes and freeways shall be 14.5 ft (4.4 m).

Falsework at traffic openings shall be protected by a temporary concrete barrier system. The falsework shall be constructed so as to provide clear distances of at least 3 in. (80 mm) between the barrier and the falsework footing and at least 1 ft (0.3 m) between the barrier and all other falsework members.

Temporary bracing required pursuant to the provisions in Section 2.3.9 shall be installed concurrently with the restrained element of the falsework system.

For falsework over or adjacent to a traffic opening, all details that contribute to horizontal stability and resistance to impact, except bolts in bracing, shall be installed at the time each element of the falsework is erected and shall remain in place until the falsework is removed.

2.5.9 Adjustment

2.5.9.1 Wedges

Wedges shall be installed in sets of two wedges (matched pair) except that a single wedge may be used on a sloping surface. Wedges shall have a height not exceeding one-third of their length. When installed, wedge sets shall be in contact over at least half of their sloping faces.

Wedges may be used at either the top or bottom of a post or strut, but not at both ends.

2.5.9.2 Jacks

Screw jacks shall not be extended beyond the limit set by the manufacturer.

Where hydraulic jacks are used for adjustment, the load imposed by the supported member shall be transferred at the end of the adjustment cycle to a permanent means of support capable of resisting the load without additional settlement or distortion.

Where sand jacks are used, the annular distance between the confining element of the jack and the edge of the base plate shall not exceed ¼ in. (6 mm).

2.5.10 Camber Strips

When directed by the owner, camber strips shall be furnished and installed to compensate for beam deflection, vertical alignment, and anticipated structure deflection.

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2.5.11 Loading

Control of the sequence and rate of placing of concrete shall be exercised to minimize unbalanced load conditions. The concrete shall also be discharged onto the formwork in a manner that prevents localized overloading.

2.5.12 Removal

Unless otherwise specified or approved, falsework shall be released before the railings, copings, or barriers are placed for all types of bridges. For arch bridges, the time of falsework release relative to the construction of elements of the bridge above the arch shall be as shown on the plans or directed by the Engineer.

Falsework supporting any span of a continuous or rigid frame bridge shall not be released until the compressive strength requirements have been satisfied for all of the structural concrete in that span and in the adjacent portions of each adjoining span for a length equal to at least one-half the length of the span.

For post-tensioned concrete bridges, the falsework supports that might continue to remain engaged after structural units are prestressed shall be released in such a manner as to permit the concrete to accept its own weight and distribute stresses gradually. Sequence of disengagements shall be such that fixed connections at tops of piers will not be subjected to damaging forces. Sequence of falsework support disengagement shall be shown on falsework drawings.

2.5.13 Dismantling

The falsework shall be designed with due regard for ease and safety of dismantling. Suitable adjustment devices shall be provided for alignment of the falsework during erection, and to facilitate dismantling. Sections of the falsework that are to be shifted and reused without dismantling shall be designed to resist the forces imposed on the falsework during moving operations.

Section 3

FORMWORK

3.1 MATERIALS AND FORM ACCESSORIES

3.1.1 General

Concrete forms shall be mortartight, true to the dimensions, lines, and grades of the structure, and of sufficient strength to prevent appreciable deflection during the placing of the concrete.

Prior to the use of each forming system to be used for exposed surfaces and when requested by the Engineer, the Contractor shall furnish form design and materials data to the Engineer for approval.

3.1.2 Sheathing

Form panels for exposed surfaces shall be plywood, conforming to or exceeding the requirements of U.S. Product Standard PS-1 for Exterior B-B (concrete form) Plyform Class I, Structural Class I or other equivalent grades of plywood, used with the face grain perpendicular to the joists. Overlaid, laminated, or other types of form panels may be utilized at the Contractor's discretion, provided that the physical properties conform to requirements of the formwork design. They shall be free of knotholes, warps, or other defects.

3.1.3 Structural Supports

Materials used for structural supports shall conform with the applicable provisions of Section 2.1.

Vertical side forms, wall forms and column forms and their related studs, walers, etc. are all defined as formwork. Structural supports on the soffit of a bridge deck and slab overhangs are falsework by definition and shall be designed accordingly.

3.1.4 Prefabricated Formwork

The Contractor shall furnish shop drawings and technical data substantiating load-carrying capacity and detailing application instructions and limitations of use. The Contractor shall utilize prefabricated product in accordance with manufacturer's recommendations.

3.1.5 Stay-in-Place Formwork

Stay-in-place soffit forms, such as corrugated metal or precast concrete panels, may be used if approved by the Engineer. If the use of such forms is proposed by the Contractor, complete details for their use shall be provided to the Engineer for review and approval prior to use. The detailed plans for structures, unless otherwise noted, are dimensioned for the use of removable forms. Any changes necessary to accommodate stay-in-place forms, if approved, shall be at the expense of the Contractor.

Stay-in-place steel bridge deck forms and supports shall be fabricated from steel conforming to ASTM Specification A446 Grade A-3 and having a coating class of G165, according to ASTM Specification A525.

Stay-in-place precast bridge deck forms shall be manufactured in accordance with *Recommended Practice for Precast Prestressed Concrete Composite Bridge Deck Panels.*[18]

3.1.6 Form Accessories

When form ties, form hangers, anchor ties, column clamps, inserts, and other similar devices are used to support formwork, the allowable working load shall be based on the in situ load conditions. When requested by the Engineer, the Contractor shall provide shop drawings and technical data substantiating the load-carrying capacity and detailing application instructions and limitations of use. In all cases, the Contractor is required to follow the manufacturer's recommended instructions.

3.2 LOADS

3.2.1 Vertical Loads

The total dead load shall equal the weight of the formwork plus the weight of the freshly placed concrete.

The minimum live load shall be 50 psf (2,400 N/m²) for the vertical load of construction traffic. This requirement is for formwork only, and does not apply to the underlying falsework. When large equipment is to be utilized during the construction process, including motorized carts, the minimum live load shall be 75 psf (3,600 N/m²).

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The minimum design load for combined dead and live loads shall not be less than 100 psf (4,800 N/m^2), or 125 psf (6,000 N/m^2) if motorized carts are used.

3.2.2 Lateral Pressure of Fluid Concrete

3.2.2.1 Unless the conditions of Section 3.2.2.2 apply, formwork shall be designed for the lateral pressure of the newly placed concrete given by Equation 3-1.

$$p = (w)(h) \qquad (3\text{-}1)$$

where p = lateral pressure, psf;
w = unit weight of fresh concrete, pcf;
h = depth of fluid or plastic concrete, ft.

For columns, or other forms that may be filled rapidly before any stiffening of the concrete takes place, h shall be taken as the full height of the form. It shall be the distance between construction joints when more than one placement of concrete is to be made.

3.2.2.2 For concrete made with Type I cement, weighing approximately 150 pcf (23.6 kN/m^3), containing no pozzolans or admixtures and having a slump of 4 in. (100 mm) or less, formwork shall be designed for a lateral pressure as follows:

For columns:

$$p = 150 + 9{,}000\, R/T \qquad (3\text{-}2)$$

with a maximum of 3,000 psf, a minimum of 600 psf, but in no case greater than 150 h.

For walls with a rate of placement less than 7 ft per hour:

$$p = 150 + 9{,}000\, R/T \qquad (3\text{-}3)$$

with a maximum of 2,000 psf, a minimum of 600 psf, but in no case greater than 150 h.

For walls with a rate of placement of 7 to 10 ft per hour:

$$p = 150 + 43{,}400/T + 2{,}800\, R/T \qquad (3\text{-}4)$$

with a maximum of 2,000 psf, a minimum of 600 psf, but in no case greater than 150 h.

where R = rate of placement, ft per hour;
T = temperature of concrete in the form, °F.

3.2.3 Horizontal Loads

Vertical wall or side form bracing shall be designed to meet the minimum wind load requirements of ASCE 7-88 (formerly ANSI A58.1) or the local building code, whichever is more stringent. For wall forms exposed to the elements, the minimum wind design load shall not be less than 15 psf (720 N/m^2). Bracing for wall forms shall be designed for a horizontal load of at least 100 plf (1,500 N/m) of wall, applied at the top.

3.3 DESIGN

3.3.1 General

The design of formwork shall be the responsibility of the Contractor. When required by the Contract Documents or pursuant to Section 3.1.1, shop drawings shall be submitted for review by the Engineer. Such review does not relieve the Contractor of the responsibility of constructing the structure in accordance with the Contract Documents.

In selecting the hydrostatic pressure to be used in the design of forms, consideration shall be given to the maximum rate of concrete placement to be used, the effects of vibration, the temperature of the concrete, and any expected use of set-retarding admixtures or pozzolanic materials in the concrete mix.

3.3.2 Allowable Stresses

Unit stresses for use in the design of formwork, exclusive of accessories, shall conform with applicable codes or standards. When fabricated formwork, shoring, or scaffolding units are used, manufacturer's recommendations for allowable loads may be followed if supported by the test reports of a qualified and recognized testing agency. For formwork materials that will experience substantial reuse, reduced values shall be used.

3.3.3 Deflection

Forms for exposed concrete surfaces shall be designed and constructed so that the formed surface of the concrete does not undulate excessively in any

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direction between studs, joists, form stiffeners, form fasteners, or wales. Undulations exceeding either 1/8 in. (3.2 mm) or 1/240 of the center-to-center distance between studs, joists, form stiffeners, form fasteners, or wales will be considered to be excessive. Should any form or forming system, even though previously approved for use, produce a concrete surface with excessive undulations, its use shall be discontinued until modifications satisfactory to the Engineer have been made. Portions of concrete structures with surface undulations in excess of the limits herein may be rejected by the Engineer.

3.3.4 Safety Factors for Form Accessories

Minimum factors of safety for formwork accessories such as form ties, form anchors, and form hangers shall conform with the safety factors presented in Table 3.1. In selecting these accessories, the formwork designer shall make certain that materials furnished for the job meet these minimum strength safety requirements.

3.4 CONSTRUCTION

3.4.1 General

All forms shall be constructed and maintained in a mortar-tight condition, including compensation for lumber shrinkage. Formwork shall be constructed and erected in such a manner as to prevent injury and to facilitate stripping and removal. Prior to concrete placement, specified formliners shall be placed and inspected to ensure proper positioning. Where required, formwork joints shall be filled with an approved material that is impervious to moisture, will not stain concrete, and produces a tight joint.

Exposed outside edges should be chamfered with a minimum ½ in. (13 mm) material unless otherwise shown. Bevels are permitted at formwork projections such as beam copings, girders, and other difficult intersections to facilitate formwork removal. Spreader blocks and bracing shall be removed during concrete placement.

3.4.2 Tolerances

All formwork shall be built and erected true to line and grade as specified in Contract Documents. Sufficient support systems must be designed to maintain formwork alignment during construction.

Unless specified in the Contract Documents, the formwork shall be constructed to conform with the tolerance limits of *Standard Specification for Tolerances for Concrete Construction and Materials* (ACI 117-90).[19] The class of surface shall also be specified in accordance with ACI 117 documents.

Formwork shall be inspected by the Contractor in the presence of the Engineer prior to placement of concrete. Such inspection does not relieve the Contractor of the responsibility of obtaining a concrete structure and finish within the specified tolerances, free of warping, bulging, or other defects. In the event of a defect, the repair method, including removal and replacement, must be approved by the Engineer and shall be completed at the Contractor's expense.

3.4.3 Joints

Expansion joints, construction joints, and isolation joints shall be located as shown on the Contract Documents. The location of joints that

Table 3.1 Minimum Safety Factors of Formwork Accessories[a]

Accessory	Safety factor	Type of construction
Form tie	2.0	All applications
Form anchor	2.0	Formwork supporting form weight and concrete pressures only
	3.0	Formwork supporting weight of forms, concrete, construction live loads, and impact
Form hangers	2.0	All applications
Anchoring inserts used as form ties	2.0	Precast concrete panels when used as formwork

Note:
(a) Safety factors are based on ultimate strength of accessory.

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become necessary due to equipment breakdown or other interruption must be approved by the Engineer.

Concrete shall be placed continuously from joint to joint. Reinforcement shall pass through construction joints or shall be connected at the joints with mechanical splices. Keyways, when required, shall be formed to assure a full keyway and accurate alignment.

Bulkheads shall be adequate to support the lateral concrete pressures without visible movement during concreting operations. The concrete shall be vibrated at the bulkhead joint to assure good consolidation of the concrete at the joint.

Expansion joints shall be constructed to permit free movement. Projections of concrete into the joint that are likely to spall under movement or prevent the proper operation of the joint are to be carefully removed.

3.4.4　Form Accessories

Metal ties or anchorage utilized to maintain formwork in proper alignment shall be manufactured to permit removal or breakback to a depth of at least 1 in. (25 mm) from the face of the concrete without damage to the concrete. The cavities created shall be filled entirely with a cementitious mortar. Where removable through ties are utilized, the cavity created shall be constructed as to minimize the area. The cavities shall be filled entirely with a cementitious mortar.

The working components coil rod, coil bolts, or other fastening mechanisms should have the ends that are to be encased in concrete greased or oiled to provide easy removal. All reused parts shall be thoroughly inspected for straightness, thread wear, and other types of damage.

3.4.5　Prefabricated Formwork

The requirements for prefabricated forms, regardless of material type, are the same as that required for standard formwork including design, strength, mortar tightness, chamfers, filleted corners, bracing, alignment, oiling, and reuse. The sheathing shall be of such strength that it remains true to shape and alignment. All bolted and riveted heads shall be countersunk on the face of the form.

3.4.6　Stay-in-Place Formwork

Form sheets shall not be permitted to rest directly on the top of the stringer or floor beam flanges. Sheets shall be securely fastened to form supports and shall have a minimum bearing length of 1 in. (25 mm) at each end. Form supports shall be placed in direct contact with the flange or stringer or floor beam. All attachments shall be made by permissible welds, bolts, clips, or other approved means. However, welding of form supports to flanges of steels not considered weldable and to portions of flange subject to tensile stresses shall not be permitted. Welding and welds shall be in accordance with the provisions of AWS D2.0 pertaining to fillet welds except that 1/8 in. (3 mm) fillet welds will be permitted.

Any permanently exposed form metal where the galvanized coating has been damaged shall be thoroughly cleaned, wire-brushed, and painted with two coats of zinc oxide-zinc dust primer, Federal Specification TT-P-641d, Type II, no color added, to the satisfaction of the Engineer. Minor head discoloration in areas of welds need not be touched up.

Transverse construction joints shall be located at the bottom of a flute and ¼ in. (6 mm) weep holes shall be field-drilled at not less than 12 in. (300 mm) on center along the line of the joint. Locate joint and weep holes at lowest portion of concrete soffit.

3.4.7　Bracing and Guying

Forms for walls or columns shall be guyed, shored, or braced to resist wind loads and to withstand alignment shifts resulting from construction live loads. Single-sided bracing shall be designed to withstand tension and compression forces. Wales and other form members shall be designed to transmit accumulated horizontal forces to strut bracing.

3.4.8　Form Removal

3.4.8.1　General

Forms shall not be removed without approval of the Engineer. In the determination of the time for the removal of forms, consideration shall be given to the location and character of the structure, the weather, the materials used in the mix, and other conditions influencing the early strength of the concrete.

Methods of removal likely to cause overstressing of the concrete or damage to its surface shall not be used. Supports shall be removed in such a manner as to permit the structure to uniformly and gradually take the stresses due to its own weight. For arch structures, the sequence of falsework release shall be as specified or approved.

3.4.8.2 Time of Removal

When field operations are controlled by cylinder tests, the removal of supporting forms or falsework shall not begin until the concrete is found to have the specified compressive strength, provided further that in no case shall supports be removed in less than 7 days after placing the concrete.

If field operations are not controlled by beam or cylinder tests, the following minimum periods of time, exclusive of days when the average temperature is below 40 °F (4 °C), shall have elapsed after placement of concrete before forms are removed:

Not supporting the dead weight of the concrete .... 24 hours

For interior cells of box girders and for railings .... 12 hours

If high early strength is obtained by the use of Type III or additional cement in the concrete mix, these periods may be reduced as permitted by the Engineer.

Forms shall not be removed until the concrete has sufficient strength to prevent damage to the surface.

3.4.9   Reuse of Formwork

Forms and form material may be reused after they have been inspected for damage. Damaged material shall be discarded or reconditioned. All material reused shall meet the requirements of forms regarding design, strength, mortar tightness, and alignment.

TEMPORARY RETAINING STRUCTURES

4.1 GENERAL

All excavations shall meet Federal OSHA Standards as outlined in 29CFR Part 1926, Subpart P.[20] Sloped excavations shall have side slopes no steeper than those specified in subpart P of OSHA 29CFR 1926 for the type of soil, as identified by a licensed geotechnical engineer.

Vertical-sided excavations shall be sheeted and braced as necessary to retain the earth and water pressures as specified in Section 4.3. The effects of any live load and dead load surcharges acting on the surrounding area shall also be considered in the design of the temporary retaining structure. Formulas given are for vertical walls with horizontal ground surface structure. For other cases, refer to standard textbooks in soil mechanics or NAVFAC DM-7.[21, 22]

4.2 TYPES OF RETAINING STRUCTURES

Selection of the type of retaining structure shall be based on an assessment of the type of soil, depth of cut, environmental conditions such as locations of nearby foundations, physical constraints, sensitivity of adjacent structures to movements, and ease of construction.

For excavations in stiff cohesive soils, common types of temporary retaining structures are wood sheeting, steel soldier piles and wood lagging, and steel sheet piles. In soft cohesive soils and wet granular soils, steel sheet piles are most commonly used. Soil washout shall be prevented by use of hay, geotextiles, excelsior, or similar materials. Where ground movement is to be minimized, all voids outside the lagging or sheeting shall be packed with soil, sand, gravel, or grout, as necessary.

4.3 LATERAL EARTH PRESSURES

4.3.1 Cantilever Walls

For a cantilever retaining system or walls with a single level of bracing, lateral pressures shall be computed by the Rankine or Coulomb method. For active pressure calculation, friction between the soil and the wall element shall be neglected. For passive pressure calculation by the Coulomb method, the angle of wall friction shall be less than one-third the effective angle of internal friction for the soil.

4.3.1.1 Wall Movement Necessary for Active Pressures

In order for the calculated pressures to be attained, the soil must undergo some strain, dependent upon the elastic modulus of the soil and the structural member in contact. Restraining effects of anchors and tiebacks may create higher than the basic active pressures from the Rankine or Coulomb formulae. Pressures in restrained cases may approach those from the passive pressure or at-rest pressure formulae.

Wall movements considered necessary to mobilize active pressure are tabulated below:

Soil Type	Wall Movement
Cohesionless, dense	0.001H
Cohesionless, medium to loose	0.002H to 0.004H
Cohesive, hard to stiff	0.01H to 0.02H
Cohesive, medium to soft	0.02H to 0.04H

where H = height of wall

4.3.1.2 Active Pressures

Cohesionless Soils

$$P_a = K_a \gamma H \qquad (4\text{-}1)$$

where

P_a = active earth pressure in psf;

γ = unit weight of soil in pcf; For most soils, moist unit weight ranges from 100 to 130 pcf (15.7 to 20.4 kN/m^3). Use total moist weight above the water table and buoyant unit weight below the water table.

H = depth below ground level in ft;

K_a = active earth pressure coefficient;

$= \tan^2 (45° - \phi/2)$;

ϕ = angle of internal friction which usually ranges from 26° to 30° for medium to dense soils.

Hydrostatic water pressure shall be added below the water table.

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Cohesive Soils

$$P_a = \gamma H - 2C \qquad (4\text{-}2)$$

where

γ = total unit weight in pcf, and other symbols are as defined above;

C = cohesion = $Q_u/2$;

Q_u = undrained unconfined compressive strength in psf.

In the absence of test data, cohesion "C" may be estimated to range between 20 to 25 percent of the effective overburden pressure.

Pressures given by this formula may be negative in the upper portion of the wall, depending on the value of cohesion. In such cases, minimum active pressure shall not be less than those for a cohesionless soil with an active earth pressure coefficient (K_a) of 0.25.

Mixed Soils

$$P_a = K_a\gamma H - 2C\sqrt{K_a} \qquad (4\text{-}3)$$

where the symbols have the same meaning as described above.

Minimum pressures shall not be less than those corresponding to an active earth pressure coefficient of 0.25.

4.3.1.3 At-Rest Pressures

Where ground movement is prevented, lateral pressures shall correspond to the at-rest values given by:

$$P_o = K_o\gamma H \qquad (4\text{-}4)$$

where

K_o = at-rest earth pressure coefficient, which is given empirically as follows:

Cohesionless soils: $K_o = 1 - \sin\phi'$

Cohesive soils: $K_o = 0.95 - \sin\phi'$

ϕ' = effective angle of internal friction.

For most normally consolidated clays, ϕ' ranges between 20° and 28°. In overconsolidated and compacted clays, K_o values can be higher and design

values shall be based on appropriate values from standard textbooks.

4.3.1.4 Passive Pressures

Cohesionless Soils

$$P_p = K_p\gamma H \qquad (4\text{-}5)$$

where

P_p = passive pressure;

K_p = passive earth pressure coefficient;

K_p = $(1 + \sin\phi) / (1 - \sin\phi)$ for zero angle of wall friction. For values of K_p, including the effect of wall friction, refer to charts in standard textbooks or NAVFAC DM-7.

Cohesive Soils

$$P_p = \gamma H + 2C \qquad (4\text{-}6)$$

Mixed Soils

$$P_p = K_p\gamma H + 2C\sqrt{K_p} \qquad (4\text{-}7)$$

Design values of cohesion shall be selected conservatively with a minimum factor of safety of 1.5. Effect of soil disturbance at the excavated subgrade shall also be considered. If duration of exposure is sufficient for pore pressure dissipation, design values in cohesive and mixed soils shall be based on effective strength parameters, neglecting the undrained cohesion.

Full passive pressures will be mobilized where ground movements will cause strains of 0.02H to .04H for medium to dense cohesionless soils, and 0.02H to 0.04H or more for hard to soft cohesive soils. Where smaller movements are anticipated, computed passive pressures shall be reduced appropriately.

4.3.2 Braced Excavations

For braced retaining structures with two or more levels of bracing, design shall be based on apparent earth pressure diagrams given by empirical methods or any other applicable earth pressure distribution developed for this purpose. The apparent earth pressure diagram in Figure 4.1 is reproduced from the AASHTO 1991 *Interim Specifications*.[4]

The figure for soft to medium clays shall be used when the stability number given by $\gamma H/C$ exceeds 4.0.

Using Fig. 4.1, reaction at the bracing levels shall be determined by assuming simple hinges at the bracing points and a fictitious hinge below the cut level. Penetration of sheathing shall be sufficient so that enough passive soil resistance can be developed. The lower hinge point shall depend on the strength of a soil, depth of cut, duration, and the type of retention system. For normal soils, this depth would be in the range of 2 to 6 ft (0.6 to 1.8 m) below the excavation level and may be assumed at the point of zero net pressure. Wall elements may be designed for the same pressures, assuming hinges at support points.

Alternate design methods based on active and passive pressures and staged excavations with continuous wall elements may also be used provided the continuity of wall elements, soil-structure interaction, and deflections during the various stages of excavation are considered.

4.3.3   Surcharge Pressures

In addition to soil and hydrostatic pressures, the retaining structure shall be designed to include the effect of surcharge loads acting within a 1.5 (horizontal): 1 (vertical) zone from the base of the excavation. Surcharge lateral pressures shall be evaluated using the same earth pressure coefficient as for the active soil pressures or using elastic solution charts and tables given in NAVFAC DM-7 or any standard textbook on soil mechanics.

4.4   STABILITY

Overall stability of a sloped excavation or a retained excavation shall be investigated by limit equilibrium methods or empirical methods. Charts and figures for stability analyses are available in standard textbooks in soil mechanics. A computer program based on the Simplified Bishop method or the Janbu method can also be used for stability analysis. Minimum factor of safety shall be 1.3.

For excavations in cohesive soils, stability against basal heave shall be investigated, using standard methods or empirical charts given in NAVFAC DM-7 and other textbooks on soil mechanics. Critical conditions shall be evaluated by a licensed geotechnical engineer.

Excavations in wet cohesionless soils shall be evaluated against piping or basal quick conditions by use of flow nets or charts for control of ground water

seepage, as given in NAVFAC DM-7. Similar charts are also available in standard textbooks in geotechnical engineering. In layered soils, blowup of cohesive soil layers due to hydrostatic pressure in underlying cohesionless layers shall be considered. Where conditions require, a pressure relief system shall be provided.

Variations of loadings on different sides of a cofferdam due to different ground levels and surcharge loadings shall be evaluated.

4.5   COFFERDAMS

Temporary cofferdams for construction of bridge foundations shall be designed for soil and water pressures and other loads and factors as outlined in the previous sections. Design water levels shall be specified on the drawings with provision for wave height.

Cofferdam size shall be adequate for the construction of the foundations and structure within it. Allowance in size shall be made for possible misalignment of the wall elements during their installation (driving) of sheet piles or soldier piles, presence of obstructions, and anticipated movements. In braced excavations, encroachments by walers and struts shall be considered.

4.5.1   Cantilever Walls

Cantilever systems shall have a factor of safety against overturning of at least 1.5. Figures 4.2 through 4.5, reproduced from the AASHTO 1991 *Interim Specifications,* are diagrams of active and passive pressures for analysis of cantilever systems.

Embedment depth of sheet piles in wet cohesionless soils shall be adequate to control piping with a minimum factor of safety of 1.5.

4.5.2   Braced Cofferdams

Braced cofferdams in dry conditions shall be designed as a structure to resist loads as outlined in the previous sections. The designer shall specify the design assumptions including soil parameters, ground water levels outside and inside the cofferdam during various stages of excavation, and the cut levels when braces are to be installed. For cofferdams in wet construction, any installation of bracing under water and prior to interior excavation shall be so specified. Design loads for the braces and required preloads shall be indicated on the design drawings.

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Cofferdams in wet conditions such as those for construction of foundations in a body of water shall be designed for conditions occurring during various stages of construction, considering the stability of basal soils, the cofferdam, and the structure to be constructed. Where hydrostatic uplift cannot be controlled by dewatering alone, a tremie seal shall be designed to resist hydrostatic uplift during dewatering.

Resistance of tremie seals shall be based on the weight of the seal concrete, the cofferdam elements, and foundation piles, if any, bonded to the tremie seal. Skin fictional resistance of piling and cofferdam elements (for example, sheeting) below the depth of excavation shall be evaluated based on the effective stress principles, but shall not exceed 100 psf (4,800 N/m^2).

Provision shall be made for flooding of the cofferdam at stages of water level exceeding the design water level of the tremie seal.

Cofferdams in a navigable channel shall be designed for impact from waterway traffic or suitably protected from impact. Loads imposed by work barges and from flowing water shall be included along with dynamic forces from fluctuating water level. Dynamic water pressure from flowing water shall be calculated from:

$$P_w = Kv^2 \qquad (4\text{-}8)$$

In the formula, P_w is the pressure in psf; v is the water velocity in fps; and K is a constant that shall take the following values:

> 1.375 for square faces
> 0.67 for circular piers
> 0.5 for angular faces.

These pressures are applicable for both a cantilever and a braced cofferdam.

Soil Type	Apparent Earth Pressure Distribution
Sand[(d)] (or Permanent Walls in Clay)	
Soft to Medium Clay[(d)] (q_u = 0.25 to 1.0 tsf)	
Stiff to Hard Clay[(d)] (q_u > 1.0 tsf)	
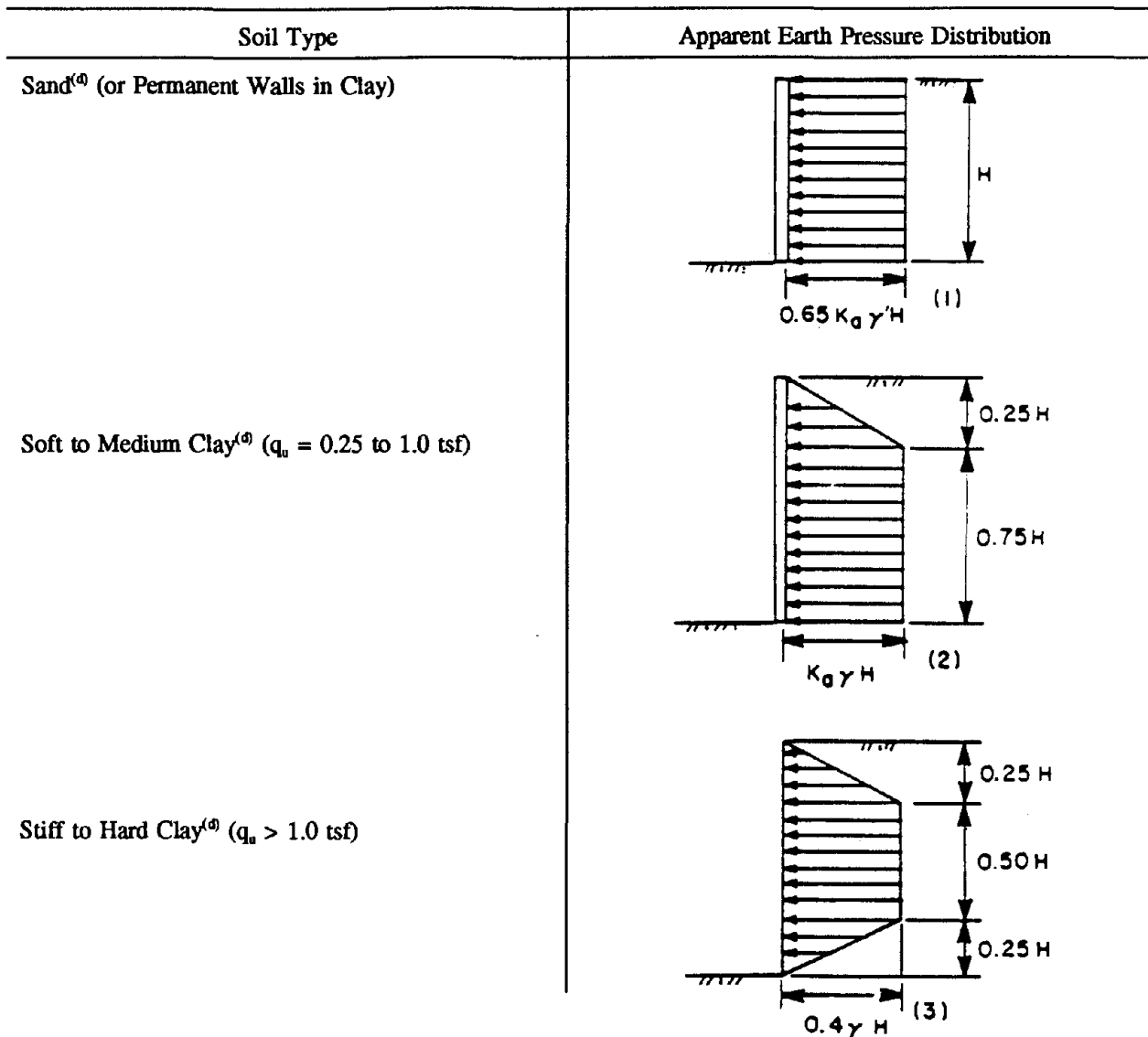
Figure 4.1 Guidelines for Estimating Earth Pressure on Walls with Two or More Levels of Anchors Constructed from the Top Down [modified after Terzaghi and Peck (1967)]

Notes:

(a) $K_a = \tan^2(45° - \phi'/2)$

(b) $K_a = 1 - m\ (2q_u/\gamma H)$, but not less than 0.25
   m = 1 for overconsolidated clay
   m = 0.4 for normally consolidated clay

(c) Value of 0.4 should be used for long-term excavations; values between 0.4 and 0.2 may be used for short-term conditions.

(d) Surcharge and water pressures must be added to these three earth pressure diagrams. The two lower diagrams are not valid for permanent walls or walls where water level lies above bottom of excavation.

(e) Conversion: 1 tsf = 96 kN/m².

Notation:

H	=	final wall height
K_a	=	active earth pressure coefficient[(a,b)]
ϕ'	=	effective angle of internal friction
γ'	=	effective soil unit weight
γ	=	total soil unit weight
m	=	reduction factor
q_u	=	unconfined compressive strength

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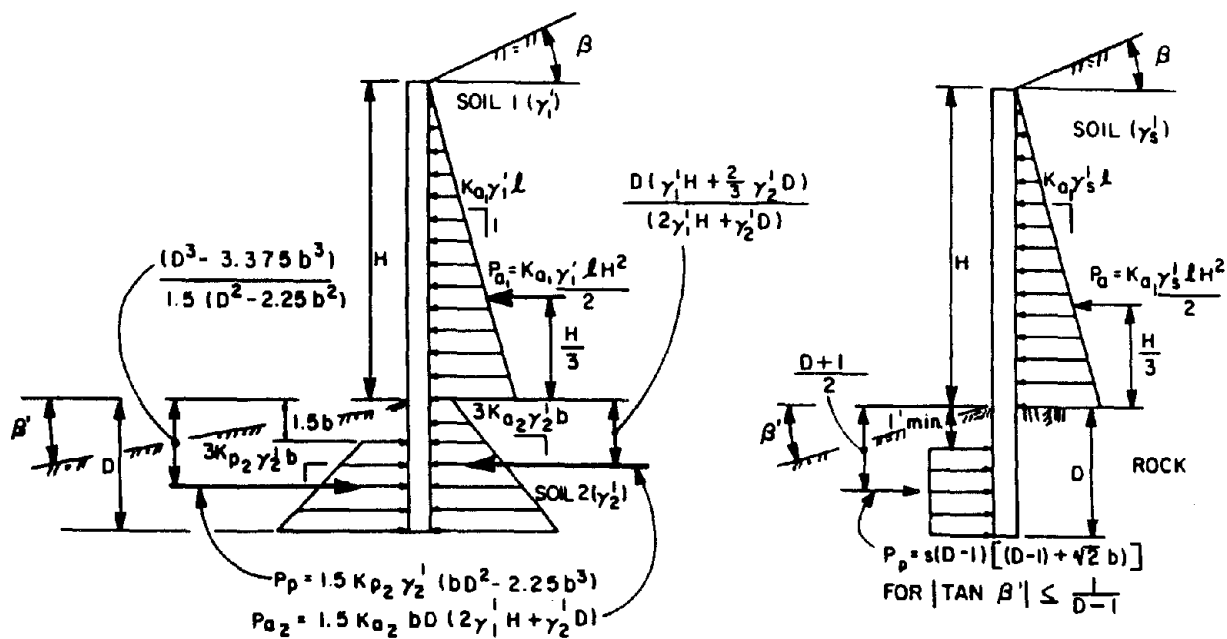
a. EMBEDMENT IN SOIL

b. EMBEDMENT IN ROCK

Figure 4.2 Simplified Earth Pressure Distributions for Permanent Flexible Cantilevered Walls with Discrete Vertical Wall Elements

Notes:

(a) For temporary walls embedded in granular soil or rock, refer to Figure 4.2 to determine passive resistance and use diagrams in Figure 4.4 to determine active earth pressure of retained soil.

(b) Surcharge and water pressures must be added to the indicated earth pressures.

(c) Forces shown are per vertical wall element.

(d) Pressure distributions below the exposed portion of the wall are based on an effective element width of 3b, which is valid for $1 \geq 5b$. For $1 < 5b$, refer to Figures 4.3 and 4.5 for continuous wall elements to determine pressured distributions on embedded portions of the wall.

(e) Refer to Reference 3 for determination of K_a and K_p.

Notation:

γ'	=	Effective unit weight of soil
b	=	Vertical element width
1	=	Spacing between vertical wall elements (c/c)
s	=	Shear strength of rock mass
P_p	=	Passive resistance per vertical wall element
P_a	=	Active earth pressure per vertical wall element
β	=	Ground surface slope behind wall {+ for slope up from wall, - for slope down from wall}
β'	=	Ground surface slope in front of wall {+ for slope up from wall, - for slope down from wall}
K_a	=	Active earth pressure coefficient
K_p	=	Passive earth pressure coefficient

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a. PRESSURE DISTRIBUTION

b. SIMPLIFIED DESIGN PROCEDURE
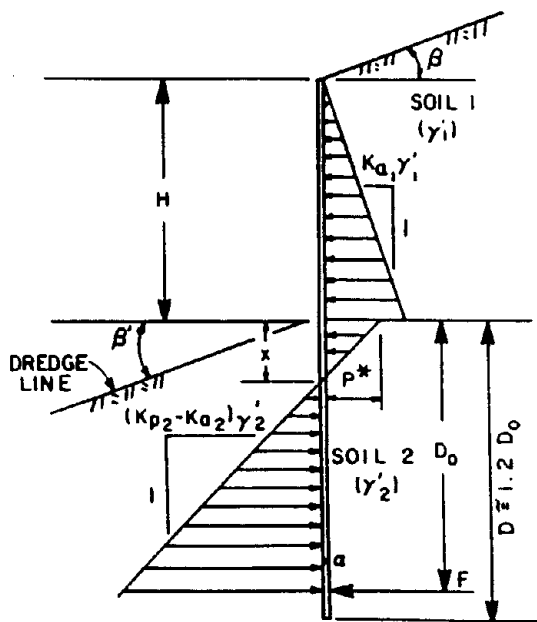
1. Determine the active earth pressure on the wall due to surcharge loads, the retained soil, and differential water pressure above the dredge line (refer to Reference 3 for determination of K_a).

2. Determine the magnitude of active pressure at the dredge line (P^*) due to surcharge loads, retained soil, and differential water pressure, using the earth pressure coefficient K_{a2}.

3. Determine the value of $x = P^*/[(K_{p2}-K_{a2})\gamma'_2]$ for the distribution of net passive pressure in front of the wall below the dredge line (refer to Reference 3 for determination of K_a and K_p).

4. Sum moments about the point of action of F to determine the embedment (D_o) for which the net passive pressure is sufficient to provide equilibrium.

5. Determine the depth (point α) at which the shear in the wall is zero (i.e., the point at which the areas of the driving and resisting pressure diagrams are equivalent).

6. Calculate the maximum bending moment at the point of zero shear.

7. Calculate the design depth, $D = 1.2D_o$ to $1.4D_o$ for a safety factor of 1.5 to 2.0.

Figure 4.3 Simplified Earth Pressure Distributions and Design Procedures for Permanent Flexible Cantilevered Walls with Continuous Vertical Wall Elements

Notes:

(a)   Surcharge and water pressures must be added to the above earth pressures.

(b)   Forces shown are per horizontal foot of vertical wall element.
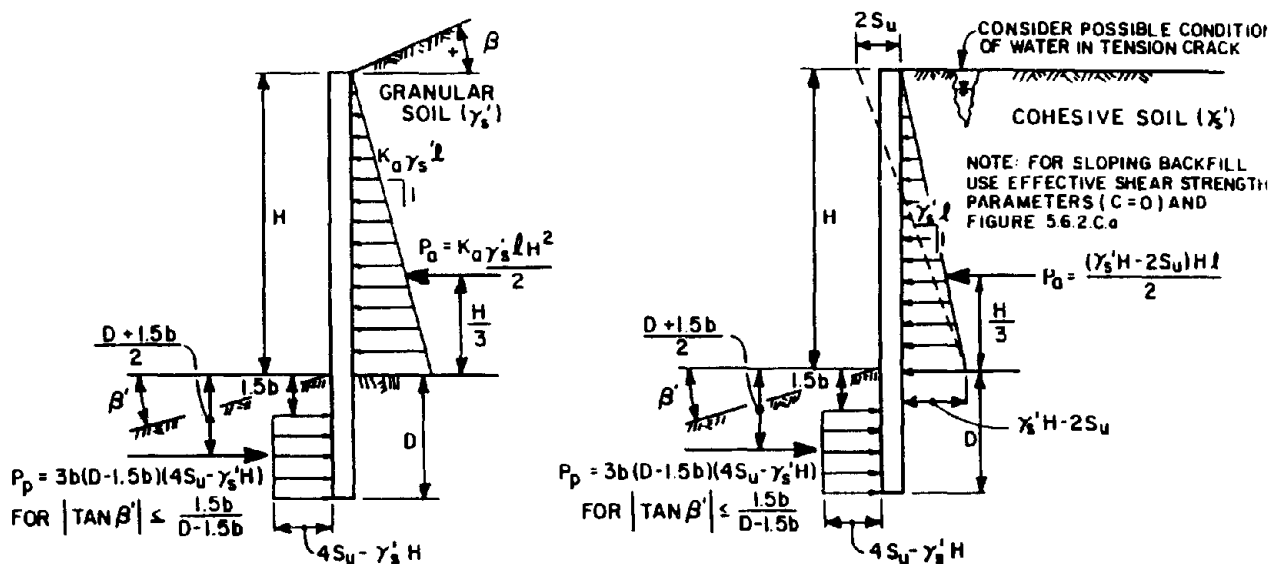
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a. EMBEDMENT IN COHESIVE SOIL
RETAINING GRANULAR SOIL

b. EMBEDMENT IN COHESIVE SOIL
RETAINING COHESIVE SOIL

Figure 4.4 Simplified Earth Pressure Distributions for Temporary Flexible Cantilevered Walls with Discrete Vertical Wall Elements

Notes:

(a)   For temporary walls embedded in granular soil or rock, refer to Figure 4.2 to determine passive resistance and use diagrams in Figure 4.4 to determine active earth pressure of retained soil.

(b)   Surcharge and water pressures must be added to the indicated earth pressures.

(c)   Forces shown are per vertical wall element.

(d)   Pressure distributions below the exposed portion of the wall are based on an effective element width of 3b, which is valid for $I \geq 5b$. For $l < 5b$, refer to Figures 4.3 and 4.5 for continuous wall elements to determine pressured distributions on embedded portions of the wall.

(e)   Refer to Reference 3 for determination of K_a.

Notation:

γ'	=	Effective unit weight of soil
b	=	Vertical element width
I	=	Spacing between vertical wall elements (c/c)
S_u	=	Undrained shear strength of cohesive soil
P_p	=	Passive resistance per vertical wall element
P_a	=	Active earth pressure per vertical wall element
β	=	Ground surface slope behind wall {+ for slope up from wall, - for slope down from wall}
β'	=	Ground surface slope in front of wall {+ for slope up from wall, - for slope down from wall}
K_a	=	Active earth pressure coefficient

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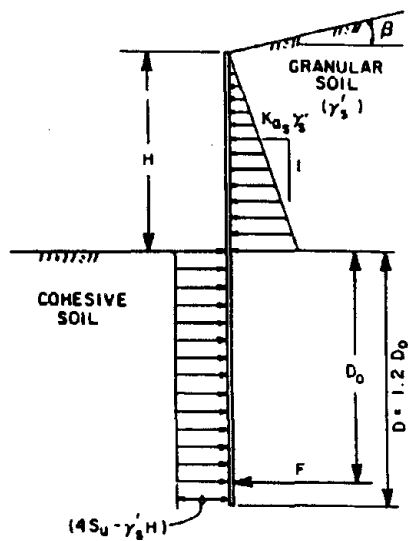
a. EMBEDMENT IN COHESIVE SOIL
RETAINING GRANULAR SOIL
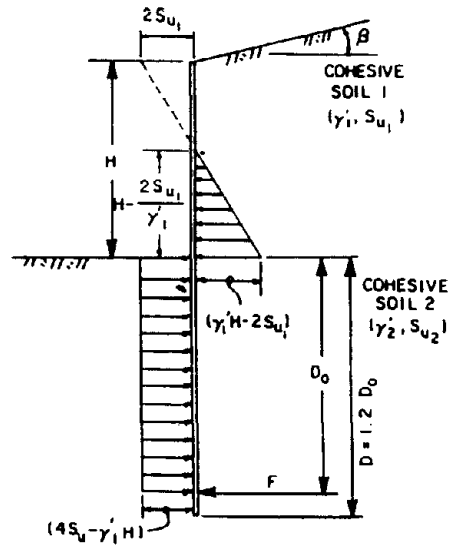
b. EMBEDMENT IN COHESIVE SOIL
RETAINING COHESIVE SOIL

Figure 4.5 Simplified Earth Pressure Distributions for Temporary Flexible Cantilevered Walls with Continuous Vertical Wall Elements [modified after Teng (1962)]

Notes:

(a)   For walls embedded in granular soil, refer to Figure 4.3 and use above diagram for retained cohesive soil when appropriate.

(b)   Refer to Figure 4.3 for simplified design procedure.

(c)   Surcharge and water pressures must be added to the above earth pressures.

(d)   Forces shown are per horizontal foot of vertical wall element.

(e)   Refer to Reference 3 for determination of K_a.

Notation:

γ'	=	Effective unit weight of soil
S_u	=	Undrained shear strength of cohesive soil
β	=	Ground surface slope behind wall {+ for slope up from wall, - for slope down from wall}
K_a	=	Active earth pressure coefficient

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GUIDE DESIGN SPECIFICATION FOR BRIDGE TEMPORARY WORKS

COMMENTARY

FALSEWORK

2.1   MATERIALS AND MANUFACTURED COMPONENTS

2.1.2   Structural Steel

The specification allows the use of both new and salvaged structural steel. Salvaged (used) steel is subject to the same ASTM A6 criteria for surface imperfections as new steel.[23] The prescribed working stresses for unknown steel grades are based on the assumed use of structural steel conforming to ASTM A36. For reference, some of the more common steel designations predating ASTM A36 are provided in Table C2.1.

Table C2.1   Early ASTM Steel Specifications[24]

Date	Specification	Remark	ASTM Requirement	
			Tensile Strength, psi	Minimum Yield Point, psi
1924-1931	ASTM A7	Structural Steel	55,000 to 65,000	½ T.S. or not less than 30,000
		Rivet Steel	46,000 to 56,000	½ T.S. or not less than 25,000
	ASTM A9	Structural Steel	55,000 to 65,000	½ T.S. or not less than 30,000
		Rivet Steel	46,000 to 56,000	½ T.S. or not less than 25,000
1939-1948	ASTM A7-A9	Structural Steel	60,000 to 72,000	½ T.S. or not less than 33,000
1939-1949	ASTM A141-39	Rivet Steel	52,000 to 62,000	½ T.S. or not less than 28,000

Conversion: 1,000 psi = 6.89 N/mm²

While it is recognized that many provisions of the *Structural Welding Code, AWS D1.1-92* may not be applicable to falsework construction, the intent of Section 2.1.2.3 is to require the same quality of workmanship for temporary works as for permanent construction. The noted exceptions are the same as those found in the *AISC Specification for Structural Steel Buildings.*[25]

2.1.3   Timber

Since falsework is seldom subjected to maximum loading for more than seven days, a load duration factor of 1.25 will be applicable to most falsework designs. In the case of loads of shorter duration, such as wind, a larger factor is appropriate. A duration-of-load factor of 2.0 (impact loading) may be used in the design of the connection at the base of a falsework post adjacent to a traffic opening.

The maximum design values for ungraded structural lumber tabulated in Appendix A are based on the lowest stresses for each size classification. These apply only for normal load duration and dry surface conditions, unless noted otherwise. Refer to the *NDS Supplement-Design Values for Wood Construction* for a description of applicable adjustment factors and species designation.[26]

2.1.5   Manufactured Components

Vertical shoring systems consist of individual components that may be assembled and erected in place to form a series of internally braced steel towers of any desired height. Safe working loads for these shoring systems are generally determined empirically by full-scale load tests, where the ultimate capacity is based upon uniform and concentric loading of the tower legs. Therefore, the shoring capacity published by the manufacturer should be considered the maximum load that the shoring is able to safely support under ideal loading conditions. Horizontal loads, eccentricity due to unequal spans or an unbalanced pouring sequence, and uneven foundation settlement generally will have an adverse effect on the vertical shoring assembly, and warrant special consideration.

Manufactured assemblies are commercial products; such as jacks, hangers, brackets, and similar items; the use of which is governed by conditions or limitations imposed by the manufacturer. When approved for use, these products become an identifiable component of the falsework system.

The specifications limit the load or the deflection or both, of any commercial product to the maximum recommended or allowed by the manufacturer of the product. The manufacturer's recommendation should be shown in a catalog or design manual published by the manufacturer, or in a statement of compliance pertaining to a particular project.

2.2   LOADS

2.2.1   General

The minimum lateral load is intended to ensure that sufficient horizontal load capacity is available so that the falsework remains stable under normal conditions, when environmental lateral loads are not present. The horizontal design load should be considered separately in both the transverse and longitudinal directions.

For post-tensioned construction, it is generally recognized that redistribution of gravity loads occurs after the superstructure is prestressed. The distribution of load in the falsework after post-tensioning is dependent on factors such as spacing and stiffness of falsework supports, foundation stiffness, superstructure stiffness, and tendon profile. The amount of load redistribution can be significant and may be a governing factor in the falsework design. Accordingly, the vertical load due to dead load transfer should be added to the remaining gravity loads to obtain the total vertical load for the falsework design.

Overall temperature variations result in contraction or expansion. The induced forces must be resisted or the movement accommodated by the falsework.

2.2.3   Live Load

2.2.3.1 Construction Live Load

Construction live loads are inherently transient and, therefore, for a given falsework scheme the potential combinations of these loads should be considered.

In bridge deck construction, a concentration of live load generally occurs at or near the edge of the bridge deck during concrete placement and finishing. To account for this loading, the guide specifications include a 75 plf (1100 N/m) live load applied along the outside edge of all deck overhangs. In the case of long falsework spans, however, the application of 75 plf (1100 N/m) to falsework components below the overhang support system may be unduly conservative. Therefore, it is recommended that the 75 plf (1100 N/m) be applied as a moving load over a length of 20 ft (6 m) and positioned to produce the maximum stress in the underlying falsework component being considered.

2.2.3.2 Impact

The allowance for impact assumes normal care in the placing of structural elements. It is not intended to cover excessive impact loads resulting from dropping materials onto the falsework or dragging structural elements into position.

2.2.5   Environmental Loads

2.2.5.1   Wind

The basic reference for computation of wind load is the *Uniform Building Code*, 1991 edition.[27] Table 2.2 was developed assuming an average C_e coefficient for exposure categories B and C, $C_q = 1.3$ to 1.4 (depending upon the height zone) and $I = 1.0$. The Q-factor was adopted from Caltrans.[28] Detailed provisions for calculating the design wind pressures are reproduced in Appendix C.

Given the regional preferences for different model codes, the relevant wind provisions from ASCE 7-88 (formerly ANSI A58.1) and the BOCA National Building Code are also provided in Appendix C.[29, 30] The wind provisions in ASCE 7-88 consist of the same general equations that appear in the AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*.[31]

For heavy-duty shoring, the wind impact is assumed to be the total projected area of all the elements in the tower face normal to the applied wind. For all other falsework, the wind impact area is assumed as the gross projected area of the falsework and any unrestrained portion of the permanent structure, excluding the area between falsework posts or towers where diagonal bracing is not used.

2.3   DESIGN

2.3.3   Stability Against Overturning

For stability analysis, it is generally assumed that the horizontal design load produces a moment that acts to overturn the falsework system or element of the system under consideration. When calculating overturning moments, the horizontal design load will be applied to the falsework in accordance with the following:

- Actual loads (such as those due to construction equipment or to the concrete placing sequence) will be considered as acting at the point of application to the falsework.

- Wind loads should be considered as acting at the centroid of the wind impact area for each height zone. When wind loads govern the design, however, the horizontal design load (to be used in calculating the overturning moment) is applied in a plane at the top of the falsework post or shoring.

- All other horizontal loads, including the minimum load when the minimum load governs, should be assumed as acting in a plane at the top of the falsework posts or shoring.

The intent of Section 2.3.3 is to insure that for each load combination identified in Section 2.3.2, the righting moment shall be at least 1.2 times the overturning moment.

2.3.5   Deflection

The theoretical deflection is the deflection that would occur if all of the concrete in the bridge superstructure were to be placed in a single pour. Theoretical deflection is limited to 1/240 of the span of the falsework beam. This limiting value is included in the specifications to ensure a certain degree of rigidity in the falsework and thereby minimize distortion of the forms. Theoretical deflection (deflection under the weight of the entire superstructure) is usually greater than the actual deflection for a given falsework member.

Actual deflection is the deflection that occurs as the falsework beam is loaded. When calculating the actual deflection, it is necessary to include the weight of forms and falsework supported by the beam as well as the weight of the concrete the beam actually supports. It is also necessary to consider such factors as the sequence of construction and the depth of the bridge superstructure when two or more concrete pours are involved.

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Readily identifiable components of the deflection arise from elastic shortening of support members and foundation settlement, but additional and often more significant deflections may occur due to take-up arising from the straightening of bent sole plates, crushing of timber packers, and other causes. The magnitude of deflections arising from take-up is largely dependent on the properties of packing materials and joint details. As a general rule, however, the vertical take-up may be on the order of 1/16 in. (2 mm) for every lumber surface in contact with another wood member or steel component.

2.3.7   Steel Beam Grillages

The current AISC specifications have been extensively modified to distinguish between local web yielding, web crippling, and sidesway web buckling.[25] The current web yielding criteria correspond to the original web crippling equations, and are an indication of the load level required to yield the web steel below the top flange. The new web crippling provisions limit concentrated loads to prevent column-type buckling of the web, and the sidesway web buckling provision limits magnitudes of concentrated load to prevent the tendency of a flange to "kick-out" under heavy compression loads.

2.3.8   Proprietary Shoring Systems

In the United States, there are several manufacturers of proprietary shoring systems. However, there are no industry standards for the various components of these systems and, as a general rule, towers or components produced by different manufacturers should not be intermixed. Some other limitations or general characteristics of modular systems are as follows:

- External bracing is recommended when the height exceeds four times the least base dimension.
- Allowable leg capacities are generally reduced when the screw jacks, or extension legs, are fully extended.
- Multi-tiered towers stacked in excess of two frames high have lower allowable leg capacities than single- or double-tier towers.
- The drift characteristics of proprietary systems can vary considerably, depending upon their bracing configurations. Ladder frames exhibit the least lateral stiffness, and very little benefit is derived from the horizontal braces.

Some manufacturers allow a 4 to 1 differential leg loading between two legs of a frame, or two frames in a tower. Examples of where this type of differential leg loading could occur would be a skewed overpass where the underlying right-of-way is maintained, or the exterior shoring towers of a bridge deck supporting screed loads and the overhang falsework. Significant differential leg loads are generally discouraged however, unless substantiating data can be furnished by the manufacturer that indicate that the differential loading will not overstress the tower components.

2.3.9   Traffic Openings

The modified design load requirement is adopted from Caltrans, where experience has shown that the downward force exerted by the bridge superstructure increases after the deck concrete is placed.[7] The increased force is the result of deck shrinkage during the curing period; consequently, it will be larger at falsework bents located near the center of the bridge span than at bents near the abutments or columns. The increased force is of greater concern in the case of cast-in-place prestressed structures (which have little load-carrying capacity until tensioned) than in conventionally reinforced concrete structures.

2.4 FOUNDATIONS

2.4.1 General

Falsework foundations are generally set at shallow depths because the loading is frequently temporary and may only last for months as opposed to loadings from permanent structures that will last for years. The foundations should be designed with an adequate safety factor against failure of the supporting ground and without detrimental settlements. The ground support factors that need to be considered include:

- The properties of the various strata below foundation level:
  - shear strength
  - compressibility.
- Site and construction considerations:
  - scour of the surface deposits
  - loss of shear strength due to construction disturbance
  - settlement due to liquefaction from vibrations
  - reduced bearing capacity due to steep ground slopes or cuts adjacent to the loaded area.
- Potential problem soils:
  - collapsible soils
  - swelling and shrinking soils
  - frost-susceptible deposits
  - fill deposits.

The purpose of this section is to provide guidelines for evaluating the properties of the various strata below the foundation level, including potential problem soils. These evaluations must be done in order to properly design foundation support for falsework systems. Detailed design procedures for these factors are not presented since the methodology for making these calculations is available in textbooks or design manuals. The designer of the falsework foundation should either be familiar with these procedures or, where deemed necessary, consult with a geotechnical or foundation engineer for recommendations. Site and construction considerations are discussed in Section 2.5.2. References 21 and 22 provide simplified foundation design procedures. Additional information pertaining to the investigation and design of falsework foundations is provided in Appendix D.

2.4.2 Footings

The use of presumptive bearing values for design is based upon judgment and experience developed on a large number of projects. For most situations, the presumptive bearing pressures, as given in Table 2.4 of the specification, are conservative. However, the presumptive bearing values do not consider important factors such as foundation size and embedment, soil stress history, and stress distribution of layered soils. Thus, the presumptive soil pressures should be considered to be an upper bound value. The design bearing pressures should be based upon site specific information and a more thorough analysis. The designer should also check the local building code, since presumptive bearing values in these codes are based upon local experience.

Soil descriptions used in Table 2.4 are in accordance with the Unified Soil Classification System, briefly described in Table D.4. These values are based upon typical soil properties for various types of deposits. For the pressures listed, settlements should be in an acceptable range.

Both uniform and differential settlement can affect the adequacy of the falsework foundations. For soil Groups 1 and 2 in Table 2.4, settlement should not be a major factor. However, settlement may be significant for the soil Group 3 and a more detailed settlement analysis shall be carried out. For soil Group 4, the use of shallow foundations is generally not acceptable. Deep foundations are normally used for support of the falsework in these deposits. The amount of movement that can occur in soil Group 5 is a function of local conditions, climatic effects, and other geologic considerations. A local geotechnical engineer shall be consulted regarding these matters.

2.4.3 Pile Foundations

If the surface deposits provide insufficient bearing capacity, or the shallow foundations are predicted to deform more than allowed, it will be necessary to extend the foundations through the surface deposits to a more competent bearing stratum. If piles are to be used, the driving criteria should be selected on the basis of a wave equation analysis or an accepted driving formula. The penetration data of the last 5 ft (1.5 m) of driving should be recorded and submitted to the designer of the falsework foundations. Guidelines for the design of pile foundations are presented in Chapter 4 of AASHTO *Standard Specifications for Highway Bridges*.[3]

2.4.4 Foundations for Heavy-Duty Shoring

In any case where the maximum leg load within a given tower exceeds 30 kips (130 kN), the tower foundation should be designed and constructed to provide uniform settlement under all legs of the tower under all loading conditions. This requirement is included in the specifications to prevent distortion of the tower components as a consequence of unequal leg settlement.

The effect of unequal leg settlement becomes more severe as leg loads increase; consequently, the tower foundation design, including the method employed to ensure uniform settlement, is relatively more important when leg loads are high.

2.5 CONSTRUCTION

2.5.2 Foundations

Surface water from rainfall or other sources could cause scour around foundations, leading to loss of support. The surface water drainage should be diverted to prevent erosion or scour during the time the falsework foundations are in use.

Construction activity taking place in the vicinity of the foundation can alter the foundation support. This can occur when trenches or pits are dug adjacent to the foundation, thereby reducing lateral support of the soil below the foundation level. Heavy construction equipment can cause rutting that will disturb the soils below the foundation level, thereby weakening them and perhaps leading to unacceptable settlements. Equipment might also be stockpiled next to foundations, causing additional loading that was not considered in the design. All construction activities should be reviewed prior to being implemented so as to maintain the adequacy of the foundations.

Certain soils are susceptible to densification from vibrations and, under the right conditions, may even liquify. These vibrations can occur from construction-related operations such as the driving of piles or sheet piling. Vibrations could also occur from movement of construction equipment across the site. Soil types A, B, and D in Group 3 of Table 2.3 are most susceptible to settlement from vibrations.

2.5.4 Steel Construction

Friction between the joists and the top flange of a steel beam will provide some lateral restraint, but the amount is indeterminate. Therefore, as a general rule, friction between the joists and top flange should be neglected when investigating flange buckling.

Timber cross bracing between adjacent steel beams is commonly used for flange support in falsework construction. In this method, timber struts are set diagonally in pairs between the top flange of one beam and the bottom flange of the adjacent beam, and securely wedged into place. However, timber cross bracing alone will not prevent flange buckling because the timber struts resist only compression forces. A more effective flange support method uses steel tension ties welded, clamped, or otherwise secured across the top and bottom of adjacent beams in combination with timber cross bracing between the beams.

When beams are continuous over two or more spans of unequal length, and if an end span is considerably shorter than the adjacent span, beam uplift may occur at the end of the short span. This uplift condition (negative deflection at the end support) is an indication of system instability and must be considered in the analysis. If theoretical uplift cannot be prevented by loading the short span first, the end of the beam must be tied down or the span lengths changed.

2.5.8   Traffic Openings

The clearances from falsework over traffic cited in this section are minimums based on Article 2.4, "Highway Clearances for Underpasses" of the AASHTO *Standard Specifications for Highway Bridges*.[3] Some States routinely specify greater clearances, that is, vertical clearances of 15 ft (4.6 m) over freeways and truck routes, and horizontal clearances to include nominal shoulders. Increased horizontal clearances should be specified when indicated by traffic needs or existing roadway geometrics at specific sites. Temporary barriers or railings to protect the falsework from vehicular impacts are normally required at all locations except where traffic speeds and volumes are very low.

2.5.12   Removal

In general, all elements of the falsework system must remain in place for a specified time period or until the concrete attains a specified strength or, in the case of cast-in-place prestressed construction, until stressing is completed. However, these limitations do not apply to bracing, including cable bracing, which is installed to prevent overturning or collapse of the falsework system. Such bracing may be removed on the day following concrete placement in any case where the in-place concrete provides horizontal stability. Note that the concrete will provide horizontal stability if, in the Engineer's judgment, it is capable of transferring horizontal forces from the falsework to previously constructed elements of the bridge substructure.

Section 3

FORMWORK

Bridge formwork can be divided into two categories: vertical and horizontal formwork. Vertical formwork can be generally constructed using job-built systems or prefabricated systems. Horizontal formwork can be constructed utilizing a job-built, prefabricated, or permanent stay-in-place system. These systems are defined as:

- Job-Built - a formwork system designed and built for a specific application, most commonly using plywood and lumber.
- Prefabricated Formwork - most commonly a modular system that has the durability for multiple reuses and normally is built with plywood with a metal framing. Prefabricated formwork can be built for custom uses on special projects.
- Stay-in-Place Formwork - a formwork system designed such that the formwork is not removed after construction. This system most commonly consists of stay-in-place metal decks or precast concrete planks.

3.1. MATERIALS AND FORM ACCESSORIES

Formwork for Concrete describes the formwork materials commonly used in the United States and provides extensive related data for form design.[14] Information is also available from manufacturers and suppliers of materials. Table C3.1 indicates other specific sources of design and specification data for formwork materials. This tabulated information should not be interpreted to exclude the use of any other materials that can meet quality and safety requirements established for the completed work.

3.1.2 Sheathing

Sheathing is defined as the supporting layer of formwork closest to the concrete. It may be in direct contact with the concrete or separated from it by a form liner. Sheathing materials consist of wood, plywood, metal, or other products capable of transferring the load of the concrete to supporting members such as joists or studs.

In selecting and using sheathing materials, important considerations are: (1) strength; (2) stiffness; (3) ease of release; (4) reuse and cost per use; (5) surface characteristics imparted to the concrete such as wood grain transfer, gloss, and paintability; (6) resistance to mechanical damage, such as from vibrators and abrasion from slip forming; (7) workability for cutting, drilling, and attaching fasteners; (8) adaptability to weather and extreme field conditions, temperature, and moisture; and (9) weight and ease of handling.

3.1.5 Stay-in-Place Formwork

In areas where form removal is expensive or hazardous, the use of stay-in-place (S.I.P) forms may be desirable. The additional dead weight of the deck slab, appearance, and corrosiveness of the environment are some of the factors that should be considered when deciding if metal or concrete S.I.P. forms should be used. Some States have developed criteria for allowing the use of corrugated steel S.I.P. forms. These criteria are generally based on the FHWA Instructional Memorandum 40-3-72 (no longer an active FHWA policy), except that deflections are limited to L/240 or ¾ in. (20 mm), whichever is less.

3.1.6 Form Accessories

Since a large proportion of formwork accessories consists of proprietary equipment, the designer shall, prior to use in designs, ensure that the engineering data provided by equipment suppliers include the following:

- The basis on which the safe working loads were determined, and whether the factor of safety is based on yield load or ultimate load.
- A statement as to whether the supplier's data are based on calculations or test results.

- Instructions compatible for use and maintenance, including points that may require special attention during formwork erection or dismantling.
- Detailed information on mass, dimensions, load capacities, deflections, shear, bending moment, and torsional strength as may be applicable.
- Number of reuses before refurbishment is required.

When threaded parts are utilized in the formwork design, the designer must call out thread type and dimensions to ensure that the component parts have compatible threads. Caution is warranted when interchanging component formwork accessories from different manufacturers because most formwork accessories are designed and tested as a system. Interchanging component parts may affect load-carrying capacity or formwork connections, leading to premature failures. Manufacturers' recommended procedures and instructions must be followed in formwork design and construction.

Table C3.1 Form Materials with References for Design and Specification

Item	Principal use	Reference data
Lumber	Form framing, sheathing, and shoring	*National Design Specification for Wood Construction (NDS)*, Reference 26, 32. *Wood Handbook: Wood as an Engineering Material*, Reference 33. *Wood Engineering*, Reference 34. *Timber Construction Manual*, Reference 35.
Plywood	Form sheathing and panels	*Construction and Industrial Plywood*, Reference 36. *Concrete Forming*, Reference 37. *Plywood Design Specifications*, Reference 38.
Steel	Panel framing and bracing Heavy forms and falsework	*Manual of Steel Construction*, Reference 25. *Cold-Formed Steel Design Manual*, Reference 39.
Aluminum	Lightweight panels and framing; bracing and horizontal shoring	*Aluminum Construction Manual*, Reference 40.
Fiber or laminated paper pressed tubes or forms	Column and beam forms; void forms for slabs, beams, girders, and precast piles	
Concrete	Footings, stay-in-place forms, molds for precast units	ACI 318, Reference 41. ACI 347R, Reference 13.
Form ties, anchors, and hangers	For securing formwork against placing loads and pressures	See Table 3.1 for recommended safety factors.
Steel joists	Formwork support	*Standard Specifications, Load Tables, and Weight Tables for Steel Joists and Joist Girders*, Reference 42.
Steel frame shoring	Formwork support	*Guide to Horizontal Shoring Beam Erection Procedure for Stationary Systems*, Reference 43. *Guidelines for Safety Requirements for Shoring Concrete Formwork*, Reference 16. *Design Manual for Structural Tubing*, Reference 44.
Form insulation	Cold weather protection of concrete	ACI 306R, Reference 45.

3.2   LOADS

3.2.2   Lateral Pressure of Fluid Concrete

The lateral pressure formulas (3-2) to (3-4), adopted from ACI, are empirical equations based on data and conditions that have changed over the years, particularly admixture usage and placement rates. The pressure formulas are applicable under the following conditions:

- The concrete weighs 150 pcf (23.6 kN/m^3), is made with Type I cement, and has a slump of not more than 4 in. (100 mm). For concrete weighing other than 150 pcf (23.6 kN/m^3), the resulting pressure from the equations is multiplied by the ratio of actual unit weight to 150 pcf (23.6 kN/m^3).

- The concrete contains no admixtures or pozzolans. When a retarding admixture, or fly ash or other pozzolan replacement of cement is used in hot weather, an effective temperature less than that of the concrete in the forms should be used.

- The temperature of the concrete ranges from 40 °F to 80 °F (4.4 °C to 26.7 °C).

- The concrete is consolidated by internal vibration to a depth of 4 ft (1.2 m) or less.

- Column forms are assumed to have a maximum plan dimension of 6 ft (1.8 m). Otherwise they are classified as wall forms.

If concrete is pumped from the base of the form, the form should be designed for full hydrostatic head of concrete plus a minimum allowance of 25 percent for pump surge pressure. If there are significant restrictions to the flow of concrete being pumped from the bottom, such as large anchorages or precast elements inside the forms, pressures may be as high as the face pressure of the pump piston.

3.3   DESIGN

3.3.2   Allowable Stresses

For formwork materials with limited reuse, allowable stresses specified in the appropriate design codes or for temporary loads on permanent structures may be used. Where there will be a considerable number of formwork reuses or where formwork is fabricated from materials such as steel, aluminum, or magnesium, it is recommended that the formwork be designed as a permanent structure carrying permanent loads.

3.4   CONSTRUCTION

3.4.2   Tolerances

Dimensional tolerances for cast-in-place concrete bridges prescribed by ACI 117 are as follows:

(a)	Departure from established alignment	1 in. (25 mm)
(b)	Departure from established grades	1 in. (25 mm)
(c)	Variation from the plumb or the specified batter in the lines and surface of columns, piers, walls, and arises	
	Exposed, in 10 ft	½ in. (13 mm)
	Backfilled, in 10 ft	1 in. (25 mm)
(d)	Variations from the level or the grades indicated on the drawings in slabs, beams, horizontal grooves, and railing offsets	
	Exposed, in 10 ft	½ in. (13 mm)
	Backfilled, in 10 ft	1 in. (25 mm)

(e)	Variation in cross-sectional dimensions of columns, piers, slabs, walls, beams, and similar parts	
	Minus	¼ in. (6 mm)
	Plus	½ in. (13 mm)
(f)	Variation in thickness of bridge slabs	
	Minus	1/8 in. (3 mm)
	Plus	¼ in. (6 mm)
(g)	Variation in the sizes and locations of the slab and wall openings	½ in. (13 m)

ACI Committee 347 defines four classes of formed surfaces, shown in Table C3.2. Class A is suggested for surfaces prominently exposed to public view, where appearance is of special importance. Class B is intended for coarse-textured concrete formed surfaces intended to receive plaster, stucco, or wainscoting. Class C is a general standard for permanently exposed surfaces where other finishes are not specified. Class D is a minimum quality requirement for surfaces where roughness is not objectionable, usually applied where surfaces will be permanently concealed.

Table C3.2 Permitted Irregularities in Formed Surfaces
(Checked with a 5-ft Template)[13]

Type of Irregularity	Class of Surface			
	A	B	C	D
Gradual	1/8 in.	¼ in.	½ in.	1 in.
Abrupt	1/8 in.	¼ in.	¼ in.	1 in.

Conversion: 1 in. = 25.4 mm, 1 ft = 0.3048 m.

3.4.3   Joints

In continuous bridges, with center spans that exceed 150 ft (45.7 m), the positive and negative moment areas should be poured separately, with a transverse construction joint installed near the point of dead load counterflexure. The placing sequence should be determined on an individual basis and shown on the Contract Documents. Positive moment regions should be placed first to avoid separation of the deck at the construction joint and to limit deck cracking in the negative moment region.

On wide bridges, a longitudinal bonded construction joint may be placed at the edge of an intermediate traffic lane. Placement of a joint within a traffic lane should be avoided. The joint should be located within the center half of the deck slab.

3.4.6   Form Removal

Formwork removal is generally specified in terms of concrete maturity or concrete strength, or a combination of both. ACI Committee 347 recommends the following with respect to time of stripping bridge formwork and its supports.[13]

Shoring and centering removal: should follow recommended practices in Sections 3.5 and 3.7 (see committee report). In no case should supporting forms and shores be removed from horizontal members before concrete strength is at least 70 percent of design strength, as determined by field-cured cylinders or other approved methods, unless removal has been approved by the Engineer. In continuous structures, support should not be released in any span until the first and second adjoining spans on each side have reached the specified strength.

Form removal: forms for ornamental work, railings, parapets, and vertical surfaces that require a surface finishing operation should be removed not less than 12 hours, nor more than 48 hours, after casting the concrete, depending on weather conditions. Bulkheads at construction joints should not be removed for a period of 15 hours after casting adjacent concrete. Forms under slab spans, beams, girders, and brackets must not be removed until the concrete has attained at least 70 percent of its design strength.

Section 4

TEMPORARY RETAINING STRUCTURES

4.1   GENERAL

Excavations required for construction of foundations and any other below-grade components of structures are made with sloping sides or with vertical or near vertical sides, depending on several factors such as available space, type of soil, water table, depth of cut, duration of the work, etc. In all cases, the conditions must provide for stability and protection of workmen as well as the newly constructed and adjacent existing structures. OSHA has specified certain minimum slopes for the various types of soils and shoring requirements for vertical-sided trenches. However, actual of site soils and neighboring conditions may require supplementary measures such as flattening of slopes, dewatering, and additional bracing.

The influence zone of surcharge pressures will depend on the type of soil. In weak soils, the zone of influence may be flatter than 45° from the base of excavation.

The formulae for lateral pressures given in Section 4.3 are for the simple case of a vertical cut in homogenous ground with a level surface. For complicated geometry, either a simplified model can be made, or an analysis can be done using the actual specific geometry and formulae given in standard textbooks.

4.2   TYPES OF RETAINING STRUCTURES

The type of retaining structure is usually selected by the Contractor. The selected scheme must satisfy the protection requirements of the constructed and existing facilities and, of course, the workmen. Ground water control is an essential element for maintaining the stability and support capacity of the soils. Soil retaining schemes may include soil stabilization by grouting, freezing, soil nailing, etc. Trench boxes installed in overexcavated trenches do not prevent ground movements. Their primary objective is protection of workmen and the installations within the trench box.

4.3   LATERAL PRESSURES

Charts and graphs are available in many textbooks for active and passive earth pressure coefficients related to the soil angle of internal friction, angle of friction between the wall element (for example, sheetpiles) and the soil, inclination of the surcharge or backfill, and the inclination of the wall. Various design simplifications are often made in the surcharge geometry to utilize the available earth pressure coefficient charts.

4.3.1.1 Wall Movement Necessary for Active Pressures

For very rigid retaining structures, ground movements may not be sufficient to mobilize active pressures. In such cases, the design should consider the higher at-rest pressures. If the retaining wall is restrained by prestressed anchors, high pressures are usually induced at the anchor location. Earth pressure distribution would be irregular and different from the usual triangular pattern of active or at-rest state. Such cases should be designed by an experienced geotechnical engineer.

The wall movements given in the specification are for rotation about the bottom of the excavation. If the top of the retaining structure is restrained and it acts as the fulcrum for rotation, then the earth pressure distribution is no longer triangular and a higher pressure occurs at the location of the support.

4.3.1.2 Active Pressure

Cohesionless Soils: The angle of internal friction for a cohesionless soil may be estimated from standard penetration tests and charts given in many soil mechanics textbooks and also in NAVFAC DM-7.[22] Careful evaluation of loose layers or zones is recommended because they may act as the weak zone of a potential slide.

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Cohesive Soils: Tension cracks and fissures, etc. in cohesive soils can destroy the cohesion. Cohesion is also lost from exposure to weather, pore pressure dissipation, erosion, remolding, and other construction activities. Hence, a minimum active pressure coefficient of 0.25 has been specified. Where free ground water can exist, earth pressures should be determined for the buoyant unit weight of the soil below the water table, and separate hydrostatic pressures added to the submerged soil pressures to determine the total earth pressure.

Mixed Soils: The above commentary for cohesive soils is also applicable to the case of mixed soils.

4.3.1.3 At-Rest Pressures

Many published empirical formulas are available for at-rest earth pressure coefficient. The selected value should be consistent with local practice. Soil type and stress history have a significant effect on this coefficient. Hydrostatic pressures should be added below the water table.

4.3.1.4 Passive Pressures

Cohesionless Soils: Passive pressure coefficients for various geometries and angles of internal friction and wall friction can be obtained from charts given in many soil mechanics textbooks and handbooks. In computing the passive pressures below the water table, use buoyant unit weight of the soil and add hydrostatic pressure. The angle of internal friction for the soil should be estimated conservatively. Charts are available in textbooks that relate the angle of internal friction to the relative density or standard penetration test values. Seepage gradients will reduce the passive pressure.

Cohesive Soils: The formula given for the passive pressure is for undrained conditions. Pore pressure dissipation can reduce the passive resistance significantly. For long duration exposure, passive pressures should also be evaluated for the drained effective friction angles using the formulas for cohesionless soils and the design based on the lower pressures. Ground water table conditions must also be considered in the analysis of earth pressures.

Mixed Soils: The above commentary for cohesionless and cohesive soils is also applicable to the mixed soils. Hydrostatic pressures in cohesionless layers interbedded in cohesive layers can cause uplift and heaving and reduce passive pressures significantly. The design should consider relief of uplift pressures, as appropriate.

4.3.2 Braced Excavations

Bracing should be designed for various stages of the excavation, corresponding to the cuts necessary to install the braces at each level. The uppermost stage is analyzed for a cantilever condition, using active pressures. The next stage with a single brace is also analyzed using active pressures and passive pressures. The braced case empirical diagrams are generally used for conditions of two or more levels of bracing.

The location of the hinge point for determining reactions at bracing levels may be estimated from the position of net zero pressure (where the passive pressure equals the active or the empirical design pressure). The wall elements must have sufficient embedment below the assumed hinge position to obtain passive reaction greater than the required toe reaction.

The design of braces should include allowances for thermal changes from ambient temperatures, misalignments, and impact from construction activities.

4.3.3 Surcharge Pressures

Surcharges near an excavation are varied in form (for example, existing foundations, proposed foundations, embankments, and construction equipment). Ground pressure on the crane tracks or pads vary during the operation of the crane. Surcharge effects should be analyzed for the most severe loading conditions. Charts are available in many soil mechanics textbooks and handbooks for analysis of lateral pressure due to various configurations of loadings.

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4.4   STABILITY

Where simplified analyses indicate critical stability conditions, methods for improvement of the soil conditions should be implemented. These methods depend on soil, ground water, depth of cut, and many other factors. Consultation with experienced geotechnical engineers is recommended for such a condition.

4.5   COFFERDAMS

The shape and size of a cofferdam is usually selected by the Contractor. The minimum size should be sufficient to construct the specified foundation. If piled foundations are to be constructed, strut and waler locations should be adjusted to allow installation of the piles.

4.5.1   Cantilever Walls

Assumptions made in the design of cantilever walls should be shown on the drawings. This is particularly important for surcharges, depths of cuts, water table, and dewatering requirements. In the case of cohesive soils, passive pressure on the embedded portion of the wall shall not exceed $2S_u$ in the case that the active pressure at the cut level on the soil side (i.e., $\gamma H - 2C$) is negative.

4.5.2   Braced Cofferdams

Design assumptions should be clearly indicated on the drawings so that construction personnel do not violate the assumptions, and variations from the design assumptions are approved by the Engineer.

APPENDIX A - MAXIMUM DESIGN VALUES FOR UNGRADED STRUCTURAL LUMBER[a]

Species/Size Classification	Extreme Fiber in Bending F_b	Tension Parallel to Grain F_t	Horizontal Shear F_v	Compression Perpendicular to Grain $F_{c\perp}$	Compression Parallel to Grain F_c	Modulus of Elasticity (psi) E
DOUGLAS FIR-LARCH						
Light Framing[b]	275	175	95	625	750	1,300,000
Joists and Planks[c]	500	325	95	625	750	1,400,000
Beams and Stringers[d]	875	425	85	625	600	1,300,000
Posts and Timbers[e]	700	475	85	625	475	1,300,000
HEM-FIR						
Light Framing[b]	250	150	75	405	850	1,100,000
Joists and Planks[c]	500	300	75	405	725	1,200,000
Beams and Stringers[d]	675	325	70	405	475	1,100,000
Posts and Timbers[e]	525	350	70	405	375	1,100,000
MIXED MAPLE						
Light Framing[b]	225	125	100	620	325	900,000
Joists and Planks[c]	400	250	100	620	325	1,000,000
Beams and Stringers[d]	625	325	90	620	375	900,000
Posts and Timbers[e]	500	350	90	620	300	900,000
MIXED OAK						
Light Framing[b]	250	150	85	800	375	800,000
Joists and Planks[c]	475	275	85	800	375	800,000
Beams and Stringers[d]	725	375	80	800	450	800,000
Posts and Timbers[e]	575	400	80	800	350	800,000
NORTHERN RED OAK						
Light Framing[b]	300	175	110	885	425	1,000,000
Joists and Planks[c]	550	325	110	885	425	1,200,000
Beams and Stringers[d]	875	425	105	885	500	1,000,000
Posts and Timbers[e]	700	475	105	885	400	1,000,000

APPENDIX A - MAXIMUM DESIGN VALUES FOR UNGRADED STRUCTURAL LUMBER[a] (CONT.)

Species/Size Classification	Stress (psi) Extreme Fiber in Bending F_b	Tension Parallel to Grain F_t	Horizontal Shear F_v	Compression Perpendicular to Grain $F_{c\perp}$	Compression Parallel to Grain F_c	Modulus of Elasticity (psi) E
RED MAPLE						
Light Framing[b]	275	150	105	615	400	1,200,000
Joists and Planks[c]	525	300	105	615	400	1,300,000
Beams and Stringers[d]	800	400	100	615	475	1,200,000
Posts and Timbers[e]	650	425	100	615	375	1,200,000
RED OAK						
Light Framing[b]	250	150	85	820	375	1,000,000
Joists and Planks[c]	475	275	85	820	375	1,100,000
Beams and Stringers[d]	725	375	80	820	450	1,000,000
Posts and Timbers[e]	575	400	80	820	350	1,000,000
SOUTHERN PINE						
Light Framing[b]	300	175	90	480	975	1,300,000
Joists and Planks[c]	575	325	90	480	825	1,400,000
Beams and Stringers[d]	1100	725	95	375	625	1,400,000
Posts and Timbers[e]	1100	725	95	375	625	1,400,000
WHITE OAK						
Light Framing[b]	250	150	110	800	400	800,000
Joists and Planks[c]	475	275	110	800	400	800,000
Beams and Stringers[d]	750	375	105	800	475	800,000
Posts and Timbers[e]	600	400	105	800	400	800,000

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Notes:
(a)   Tabulated values are generally based on the lowest stresses for each size classification and apply only for normal load duration and dry surface conditions unless noted otherwise. Refer to NDS for a description of applicable adjustment factors and species designation.
(b)   Lumber nominally 2 in. to 4 in. thick, and a maximum of 4 in. wide. Tabulated values shall be adjusted for all applicable factors given in NDS Table 4A, except for size.
(c)   Lumber nominally 2 in. to 4 in. thick, and at least 5 in. wide. Tabulated values shall be adjusted for all applicable factors given in NDS Table 4A, except for size.
(d)   Lumber of rectangular cross section, nominally at least 5 in. thick with a width more than 2 in. greater than the thickness (e.g., 6 x 10, 8 x 14). Refer to NDS Part IV for restrictions related to use of resawn, cantilevered, continuous, and flatwise use of Beams and Stringers.
(e)   Lumber of approximately square cross section, nominally at least 5 in. square with the larger dimension not more than 2 in. greater than the smaller dimension (e.g., 6 x 6, 6 x 8, 12 x 14).
(f)   Conversion: 1 ksi = 6.89 N/mm², 1 in. = 25.4 mm

APPENDIX B

AISC PROVISIONS FOR WEBS AND FLANGES
UNDER CONCENTRATED FORCES

K1.1   Design Basis

Members with concentrated loads applied normal to *one flange* and symmetric to the web shall have a flange and web proportioned to satisfy the local flange bending, web yielding strength, web crippling, and sidesway web buckling criteria of Sections K1.2, K1.3, K1.4, and K1.5. Members with concentrated loads applied to *both flanges* shall have a web proportioned to satisfy the web yielding, web crippling, and column web buckling criteria of Sections K1.3, K1.4, and K1.6.

Where pairs of stiffeners are provided on opposite sides of the web, at concentrated loads, and extend at least half the depth of the member, Sections K1.2 and K1.3 need not be checked.

For column webs subject to high shears, see Section K1.7; for bearing stiffeners, see Section K1.8.

K1.2   Local Flange Bending

A pair of stiffeners shall be provided opposite the tension flange or flange plate of the beam or girder framing into the member when the thickness of the member flange t_f is less than:

$$0.4 \sqrt{\frac{P_{bf}}{F_{yc}}} \qquad\qquad (K1\text{-}1)$$

where F_{yc} = column yield stress, ksi;

P_{bf} = the computed force delivered by the flange or moment connection plate multiplied by 5/3, when the computed force is due to live and dead load only, or by 4/3, when the computed force is due to live and dead load in conjunction with wind or earthquake forces, kips.

If the length of loading measured across the member flange is less than $0.15b$, where b is the member flange width, Equation K1-1 need not be checked.

K1.3   Local Web Yielding

Bearing stiffeners shall be provided in beams and welded plate girders if the compressive stress at the web toe of the fillets resulting from concentrated loads exceeds $0.66F_y$.

a.   When the force to be resisted is a concentrated load producing tension or compression, applied at a distance from the member end that is greater than the depth of the member:

$$\frac{R}{t_w(N + 5k)} \le 0.66F_y \qquad\qquad (K1\text{-}2)$$

b.   When the force to be resisted is a concentrated load applied at or near the end of the member:

$$\frac{R}{t_w(N + 2.5k)} \le 0.66F_y \qquad\qquad (K1\text{-}3)$$

where R = concentrated load or reaction, kips;

t_w = thickness of web, in.;

N = length of bearing (not less than k for end reactions), in.;

k = distance from outer face of flange to web toe of fillet, in.

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K1.4   Web Crippling

Bearing stiffeners shall be provided in the webs of members under concentrated loads, when the compressive force exceeds the following limits:

a.   When the concentrated load is applied at a distance not less than $d/2$ from the end of the member:

$$R = 67.5t_w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{F_{yw} \, t_f / t_w} \qquad \text{(K1-4)}$$

b.   When the concentrated load is applied less than a distance $d/2$ from the end of the member:

$$R = 34t_w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{F_{yw} \, t_f / t_w} \qquad \text{(K1-5)}$$

where F_{yw} = specified minimum yield stress of beam web, ksi;
d = overall depth of the member, in.;
t_f = flange thickness, in.

If stiffeners are provided and extend at least one-half the web depth, Equations K1-4 and K1-5 need not be checked.

K1.5   Sidesway Web Buckling

Bearing stiffeners shall be provided in the webs of members with flanges not restrained against relative movement by stiffeners or lateral bracing and subject to concentrated compressive loads, when the compressive force exceeds the following limits:

a.   If the loaded flange is restrained against rotation and $(d_c/t_w)/(l/b_f)$ is less than 2.3:

$$R = \frac{6,800t_w^3}{h} \left[1 + 0.4 \left(\frac{d_c / t_w}{l/b_f} \right)^3 \right] \qquad \text{(K1-6)}$$

b.   If the loaded flange is not restrained against rotation and $(d_c/t_w)/(l/b_f)$ is less than 1.7:

$$R = \frac{6,800t_w^3}{h} \left[0.4 \left(\frac{d_c / t_w}{l/b_f} \right)^3 \right] \qquad \text{(K1-7)}$$

where l = largest laterally unbraced length along either flange at the point of load, in.;
b_f = flange width, in.;
d_c = $d - 2k$ = web depth clear of fillets, in.;
h = clear distance between flanges at the section under investigation, in.

Equations K1-6 and K1-7 need not be checked providing $(d_c/t_w)/(l/b_f)$ exceeds 2.3 or 1.7, respectively, or for webs subject to uniformly distributed load.

K1.6 Compression Buckling of the Web

A stiffener or a pair of stiffeners shall be provided opposite the compression flange when the web depth clear of fillets d_c is greater than:

$$\frac{4100t_{wc}^3\sqrt{F_{yc}}}{P_{bf}} \tag{K1-8}$$

where t_{wc} = thickness of column web, in.

K1.7 Compression Members with Web Panels Subject to High Shear

Members subject to high shear stress in the web should be checked for conformance with Section F4.

K1.8 Stiffener Requirements for Concentrated Loads

Stiffeners shall be placed in pairs at unframed ends or at points of concentrated loads on the interior of beams, girders, or columns if required by Sections K1.2 through K1.6, as applicable.

If required by Sections K1.2, K1.3, or Equation K1-9, stiffeners need not extend more than one-half the depth of the web, except as follows:

If stiffeners are required by Sections K1.4 or K1.6, the stiffeners shall be designed as axially compressed members (columns) in accordance with requirements of Section E2 with an effective length equal to 0.75h, a cross section composed of two stiffeners and a strip of the web having a width of 25t_w at interior stiffeners and 12t_w at the ends of members.

When the load normal to the flange is tensile, the stiffeners shall be welded to the loaded flange. When the load normal to the flange is compressive, the stiffeners shall either bear on or be welded to the loaded flange.

When flanges or moment connection plates for end connections of beams and girders are welded to the flange of an I- or H-shape column, a pair of column-web stiffeners having a combined cross-sectional area A_{st} not less than that computed from Equation K1-9 shall be provided whenever the calculated value of A_{st} is positive:

$$A_{st} = \frac{P_{bf} - F_{yc}t_{wc}(t_b + 5k)}{F_{yst}} \tag{K1-9}$$

where F_{yst} = stiffener yield stress, ksi;

k = distance between outer face of column flange and web toe of its fillet, if column is a rolled shape, or equivalent distance if column is a welded shape, in.;

t_b = thickness of flange or moment connection plate delivering concentrated force, in.

Stiffeners required by the provisions of Equation K1-9 and Sections K1.2 and K1.6 shall comply with the following criteria:

1. The width of each stiffener plus ½ the thickness of the column web shall not be less than 1/3 the width of the flange or moment connection plate delivering the concentrated force.
2. The thickness of stiffeners shall not be less than ½ the thickness of the flange or plate delivering the concentrated load.
3. The weld joining stiffeners to the column web shall be sized to carry the force in the stiffener caused by unbalanced moments on opposite sides of the column.

APPENDIX C

DESIGN WIND PRESSURES AND FORCES FROM
SELECTED MODEL CODES

Uniform Building Code, 1991 Edition

Basic Wind Speed

Sec. 2314. The minimum basic wind speed for determining design wind pressures shall be taken from Figure No. 23-1. For those areas designated in Figure No. 23-1 as special wind regions and other areas where local records or terrain indicate higher 50-year (mean recurrence interval) fastest-mile wind speeds, these higher values shall be the minimum basic wind speeds.

Design Wind Pressures

Sec. 2316. Design wind pressures for structures or elements of structures shall be determined for any height in accordance with the following formula:

$$P = C_e C_q q_s I \qquad (16\text{-}1)$$

where P = design wind pressure;

C_e = combined height, exposure, and gust factor coefficient as given in Table No. 23-G;

C_q = pressure coefficient for the structure or portion of structure under consideration as given in Table No. 23-H;

q_s = wind stagnation pressure at the standard height of 30 ft (9.1 m) as set forth in Table No. 23-F;

I = importance factor (assume equal to 1.0 for falsework).

Open-Frame Towers

Sec. 2319. Radio towers and other towers of trussed construction shall be designed and constructed to withstand wind pressures specified in this section, multiplied by the shape factors set forth in Table No. 23-H.

Related Tables and Figures

Table No. 23-G - Combined Height, Exposure and Gust Factor Coefficient (C_e)[a]

Height Above Average Level of Adjoining Ground (ft)	Exposure[b] D	Exposure[c] C	Exposure[d] B
0-15	1.39	1.06	0.62
20	1.45	1.13	0.67
25	1.50	1.19	0.72
30	1.54	1.23	0.76
40	1.62	1.31	0.84
60	1.73	1.43	0.95
80	1.81	1.53	1.04
100	1.88	1.61	1.13

Notes:

(a) Values for intermediate heights above 15 ft may be interpolated.

(b) Exposure D represents the most severe exposure in areas with basic wind speeds of 80 miles per hour (mph) or greater and has terrain which is flat and unobstructed facing large bodies of water over one mile or more in width relative to any quadrant of the building site.

(c) Exposure C has terrain which is flat and generally open, extending ½ mile or more from the site in any full quadrant.

(d) Exposure B has terrain with buildings, forest, or surface irregularities 20 ft or more in height covering at least 20 percent of the area extending 1 mile or more from the site.

(e) Conversion: 1 ft = 0.3048 m; 1 mph = 1.609 km/hr; 1 mile = 1.609 km.

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Table No. 23-F - Wind Stagnation Pressure (q_s) at Standard Height of 33 ft

Basic wind speed (mph)[a]	70	80	90	100	110	120	130
Pressure q_s (psf)	12.6	16.4	20.8	25.6	31.0	36.9	43.3

Notes:

(a) Wind speed from Section 2314.

(b) Conversion: 1 ft = 0.3048 m; 1 mph = 1.609 km/hr; 1 psf = 47.88 N/m².

Table No. 23-H - Pressure Coefficients (C_q)

Structure or Part Thereof	Description	C_q factor
Primary frames and systems	**Method 1** (Normal force method) Windward wall Leeward wall	0.8 inward 0.5 outward
	Method 2 (Projected area method) On vertical projected area Structures 40 ft or less in height Structures over 40 ft in height On horizontal projected area	1.3 horizontal any direction 1.4 horizontal any direction 0.7 upward

Open-frame towers [a,b]	Square and rectangular
	   Diagonal
	   Normal
	Triangular
		4.0
		3.6
		3.2

Notes:

(a) Local pressures shall apply over a distance from the discontinuity of 10 ft or 0.1 times the least width of the structure, whichever is smaller.

(b) Wind pressures shall be applied to the total normal projected area of all elements on one face.

(c) The force shall be assumed to act parallel to the wind direction.

(d) Conversion: 1 ft = 0.3048 m.

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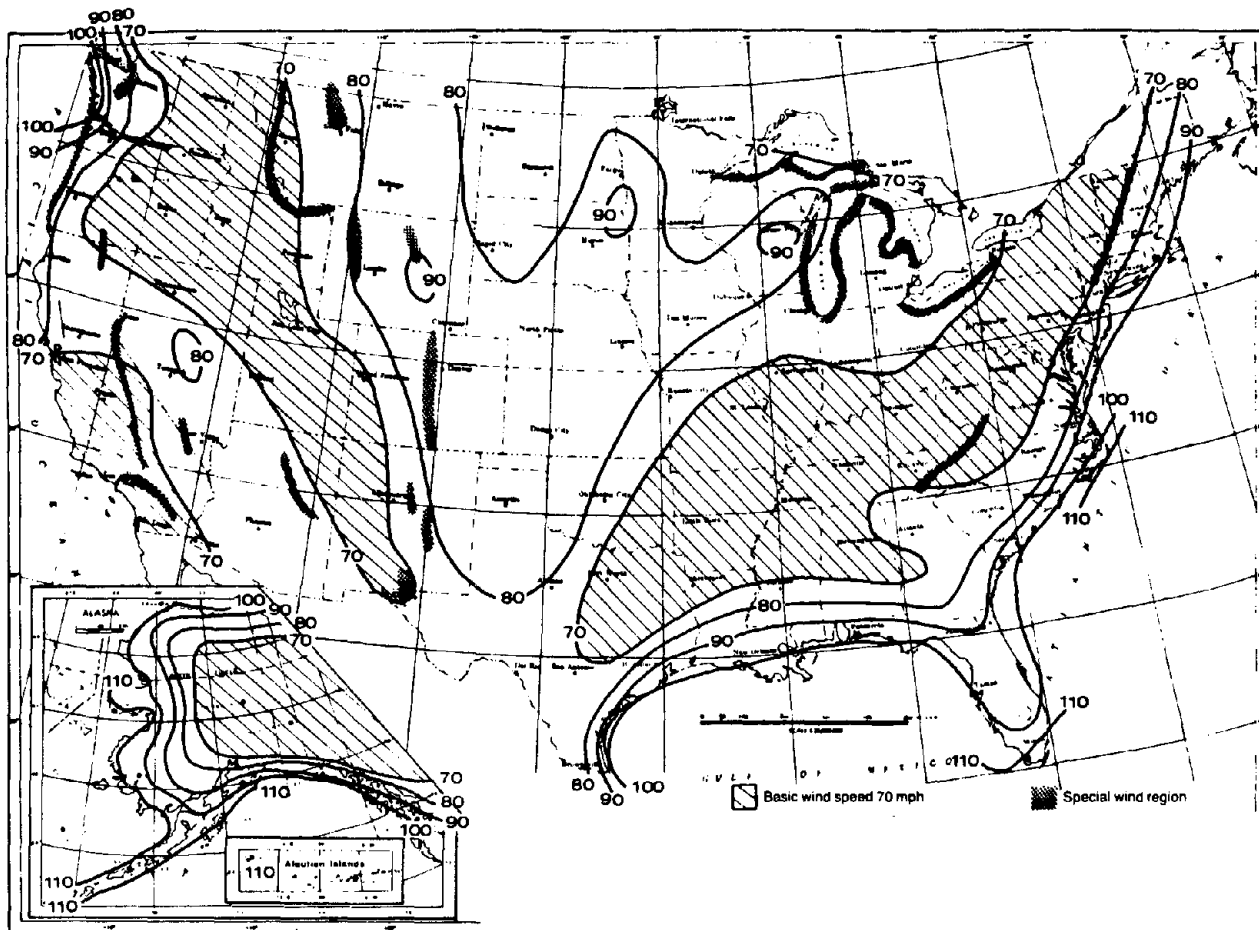
Notes:

(a) Linear interpolation between wind speed contours is acceptable.

(b) Caution in use of wind speed contours in mountainous regions of Alaska is advised.

(c) Wind speed for Hawaii is 80, Puerto Rico is 95, and the Virgin Islands is 110.

(d) Wind speed may be assumed to be constant between the coastline and the nearest inland contour.

(e) Conversion: 1 mph = 1.609 km/hr

Figure No. 23-1 - Minimum Basic Wind Speeds in Miles Per Hour

Basic Wind Speed

Definition. Fastest-mile wind speed at 33 ft (10 m) above the ground of terrain Exposure C (see 6.5.3.1) and associated with an annual probability of occurrence of 0.02.

Section 6.5.2. The basic wind speed V used in the determination of design wind loads on buildings and other structures shall be as given in Figure 1 for the contiguous United States and Alaska and in Table 7 for Hawaii and Puerto Rico except as provided in 6.5.2.1 and 6.5.2.2. The basic wind speed used shall be at least 70 mph (113 km/hr).

Design Wind Force (Table 4)

$$Design\ Wind\ Force,\ F = q_z\ G_h\ C_f\ A_f$$

where q_z (velocity pressure) = $0.00256\ K_z\ (IV)^2$;

K_z = velocity pressure coefficient as given in Table 6;

I = importance factor (assume equal to 1.0 for falsework);

V = basic wind speed obtained from Figure 1 and Table 7, in miles per hour;

G_h = gust response factor as given in Table 8;

C_f = force coefficient as given in Tables 14 and 15;

A_f = the projected area normal to the wind.

Related Tables and Figures

Table 6 - Velocity Pressure Exposure Coefficient, K_z

Height above ground level, z (ft)	K_z			
	Exposure A	Exposure B	Exposure C	Exposure D
0-15	0.12	0.37	0.80	1.20
20	0.15	0.42	0.87	1.27
25	0.17	0.46	0.93	1.32
30	0.19	0.50	0.98	1.37
40	0.23	0.57	1.06	1.46
50	0.27	0.63	1.13	1.52
60	0.30	0.68	1.19	1.58
70	0.33	0.73	1.24	1.63
80	0.37	0.77	1.29	1.67
90	0.40	0.82	1.34	1.71
100	0.42	0.86	1.38	1.75

Notes:

(a) Linear interpolation for intermediate values of height z is acceptable.

(b) Exposure categories are defined in 6.5.3.

(c) Conversion: 1 ft = 0.3048 m

Section 6.5.3   Exposure Categories

Exposure A. Large city centers with at least 50 percent of the buildings having a height in excess of 70 ft (210 m). Use of this exposure category shall be limited to those areas for which terrain representative of Exposure A prevails in the upwind direction for a distance of at least ½ mile (0.8 km) or 10 times the height of the building or structure, whichever is greater. Possible channeling effects or increased velocity pressures due to the building or structure being located in the wake of adjacent buildings shall be taken into account.

Exposure B. Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger. Use of this exposure category shall be limited to those areas for which terrain representative of Exposure B prevails in the wind direction for a distance of at least 1,500 ft (460 m) or 10 times the height of the building or structure, whichever is greater.

Exposure C. Open terrain with scattered obstruction having heights generally less than 30 ft (9 m). This category includes flat open country and grasslands.

Exposure D. Flat, unobstructed areas exposed to wind flowing over large bodies of water. This exposure shall apply only to those buildings and other structures exposed to the wind coming from over the water. Exposure D extends inland from the shoreline a distance of 1,500 ft (460 m) or 10 times the height of the building or structure, whichever is greater.

Table 7 - Basic Wind Speed, V

Location	V (mph)
Hawaii	80
Puerto Rico	95

Notes:

(a) The unique topographical features common to the islands of Hawaii and Puerto Rico suggest that it may be advisable to adjust the values given in Table 7 to account for locally higher winds for structures sited near mountainous terrain, gorges, and ocean promontories.

(b) Conversion: 1 mph = 1.609 km/hr.

Table 8 - Gust Response Factors, G_h and G_z

Height above ground level, z (ft)	G_h and G_z			
	Exposure A	Exposure B	Exposure C	Exposure D
0-15	2.36	1.65	1.32	1.15
20	2.20	1.59	1.29	1.14
25	2.09	1.54	1.27	1.13
30	2.01	1.51	1.26	1.12
40	1.88	1.46	1.23	1.11
50	1.79	1.42	1.21	1.10
60	1.73	1.39	1.20	1.09
70	1.67	1.36	1.19	1.08
80	1.63	1.34	1.18	1.08
90	1.59	1.32	1.17	1.07
100	1.56	1.31	1.16	1.07

Notes:

(a)   For main wind-force resisting systems, use building or structure height $h = z$.

(b)   Linear interpolation is acceptable for intermediate values of z.

(c)   Value of gust response factor shall be not less than 1.0.

(d)   Conversion: 1 ft = 0.3048 m.

## Table 15 - Force Coefficients for Trussed Towers, C_t

ε	C_t Square Towers	Triangular Towers
<0.025	4.0	3.6
0.025 to 0.44	$4.1 - 5.2\varepsilon$	$3.7 - 4.5\varepsilon$
0.45 to 0.69	1.8	1.7
0.7 to 1.0	$1.3 + 0.7\varepsilon$	$1.0 + \varepsilon$

Notes:

(a) The area A_f consistent with these force coefficients is the solid area of the front face projected normal to the wind direction.

(b) Force coefficients are given for towers with structural angles or similar flat-sided members.

(c) For towers with rounded members, the design wind force shall be determined using the values in the table multiplied by the following factors:

$\varepsilon \leq 0.29$, factor = 0.67

$0.3 \leq \varepsilon \leq 0.79$, factor = $0.67\varepsilon + 0.47$

$0.8 \leq \varepsilon \leq 1.0$, factor = 1.0

(d) For triangular section towers, the design wind forces shall be assumed to act normal to a tower face.

(e) For square section towers, the design wind forces shall be assumed to act normal to a tower face. To allow for the maximum horizontal wind load, which occurs when the wind is oblique to the faces, the wind load acting normal to a tower face shall be multiplied by the factor $1.0 + 0.75 \varepsilon$ for $\varepsilon < 0.5$ and shall be assumed to act along a diagonal.

(f) Wind forces on tower appurtenances, such as ladders, conduits, lights, elevators, and the like, shall be calculated using appropriate force coefficients for these elements.

(g) For guyed towers, the cantilever portion of the tower shall be designed for 125 percent of the design force.

(h) A reduction of 25 percent of the design force in any span between guys shall be made for determination of controlling moments and shears.

Notation:

ε - ratio of solid area to gross area of tower face.
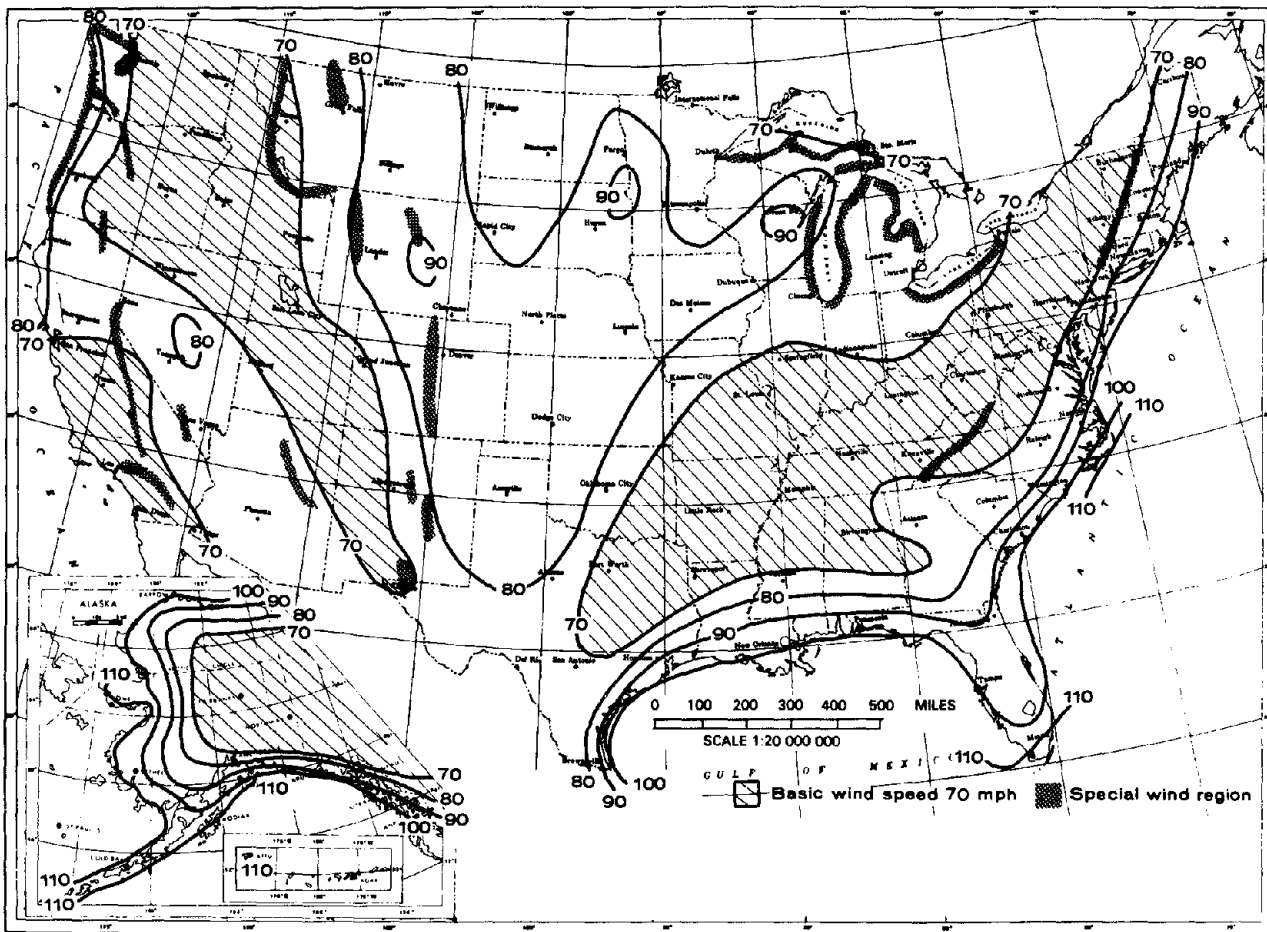
Notes:

(a)   Values are fastest-mile speeds at 33 ft (10 m) above ground for exposure category C and are associated with an annual probability of 0.02.

(b)   Linear interpolation between wind speed contours is acceptable.

(c)   Caution in the use of wind speed contours in mountainous regions of Alaska is advised.

(d)   Conversion: 1 mph = 1.609 km/hr

Figure 1 - Basic Wind Speed (mph)

BOCA National Building Code, 1990

Basic Wind Speed

Section 1112.3.2 The basic wind speed, in miles per hour, to be used for the location of the building or other structure shall be determined by Figure 1112.3.2. Basic wind speed for special wind regions indicated on Figure 1112.3.2 shall be in accordance with local jurisdiction requirements.

Design Wind Pressure

Section 1112.3 The design and wind pressure for the main windforce-resisting system shall be determined as follows:

$$P_d = P_e \, I^2 \, C_p$$

where P_e = effective velocity pressure, including gust effect as tabulated in Table 1112.3.3a(1) for Exposure B and Table 1112.3.3b for Exposure C

I = importance factor of the building or other structure (assume equal to 1.0 for falsework)

C_p = external pressure coefficient to be used in determination of *wind loads* for buildings or for other structures (see Figure 1112.2a and Tables 1112.2c through 1112.2h)

Sec. 1112.3.3 Exposure Classifications

Exposure B: Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of *single-family dwellings* or larger. Use of this exposure category shall be limited to those areas for which terrain representative of Exposure B prevails in the upwind direction for a distance of at least 1,500 ft (460 m) or 10 times the *height* of the building or structure, whichever is greater.

Exposure C: Open terrain with scattered obstructions having heights generally less than 30 ft (9.1 m). This category includes flat, open country and grasslands.

Related Tables and Figures

Table 1112.2f - FORCE COEFFICIENTS FOR TRUSSED TOWERS,[a] C_p[b],[e],[f],[g]

ε[h]	C_p	
	Square towers[d]	Triangular towers[c]
<0.025	4.0	3.6
0.025 to 0.44	4.1 - 5.2ε	3.7 - 4.5ε
0.45 to 0.69	1.8	1.7
0.7 to 1.0	1.3 + 0.7ε	1.0 + ε

Notes:

(a) Force coefficients are given for towers with structural angles or similar flat-sided members.

(b) For towers with rounded members, the design windforce shall be determined using the values in the above table multiplied by the following factors:

ε<0.29, factor = 0.67

0.3<ε<0.79, factor = 0.67ε + 0.47

0.8<ε<1.0, factor = 1.0

(c) For triangular section towers, the design windforces shall be assumed to act normal to a tower face.

(d) For square section towers, the design windforces shall be assumed to act normal to a tower face. To allow for the maximum horizontal wind load which occurs when the wind is oblique to the faces, the wind load acting normal to a tower face shall be multiplied by the factor 1.0 + 0.7ε ≤ 0.5 and shall be assumed to act along a diagonal.

(e) Windforces on tower appurtenances, such as ladders, conduits, lights, elevators and the like, shall be calculated using appropriate force coefficients for these elements.

(f) For guyed towers, the cantilever portion of the tower shall be designed for 125 percent of the design windforce.

(g) A reduction of 25 percent of the design windforce in any span between guys shall be made for determination of controlling moments and shears.

Notation:

ε - Ratio of solid area to gross area of tower.

Table 1112.3.3a - EFFECTIVE VELOCITY PRESSURES P_e(lb/ft^2)
FOR BUILDINGS AND STRUCTURES (EXPOSURE B)

Height above grade (ft)	Basic wind speed (mph)				
	70	80	90	100	110
0-20	9	12	15	18	22
20-40	10	13	17	21	25
40-60	13	16	21	26	31
60-100	14	18	23	28	34

Conversion: 1 psf = 47.88 N/m^2; 1 mph = 1.609 km/hr; 1 ft = 0.3048 m.

Table 1112.3.3b - EFFECTIVE VELOCITY PRESSURES P_e (lb/ft^2)
FOR BUILDINGS AND STRUCTURES (EXPOSURE C)

Height above grade (ft)	Basic Wind speed (mph)				
	70	80	90	100	110
0-20	15	25	25	31	37
20-40	16	27	27	33	40
40-60	19	31	31	38	47
60-100	20	33	33	41	50

Conversion: 1 psf = 47.88 N/m^2; 1 mph = 1.609 km/hr; 1 ft = 0.3048 m.
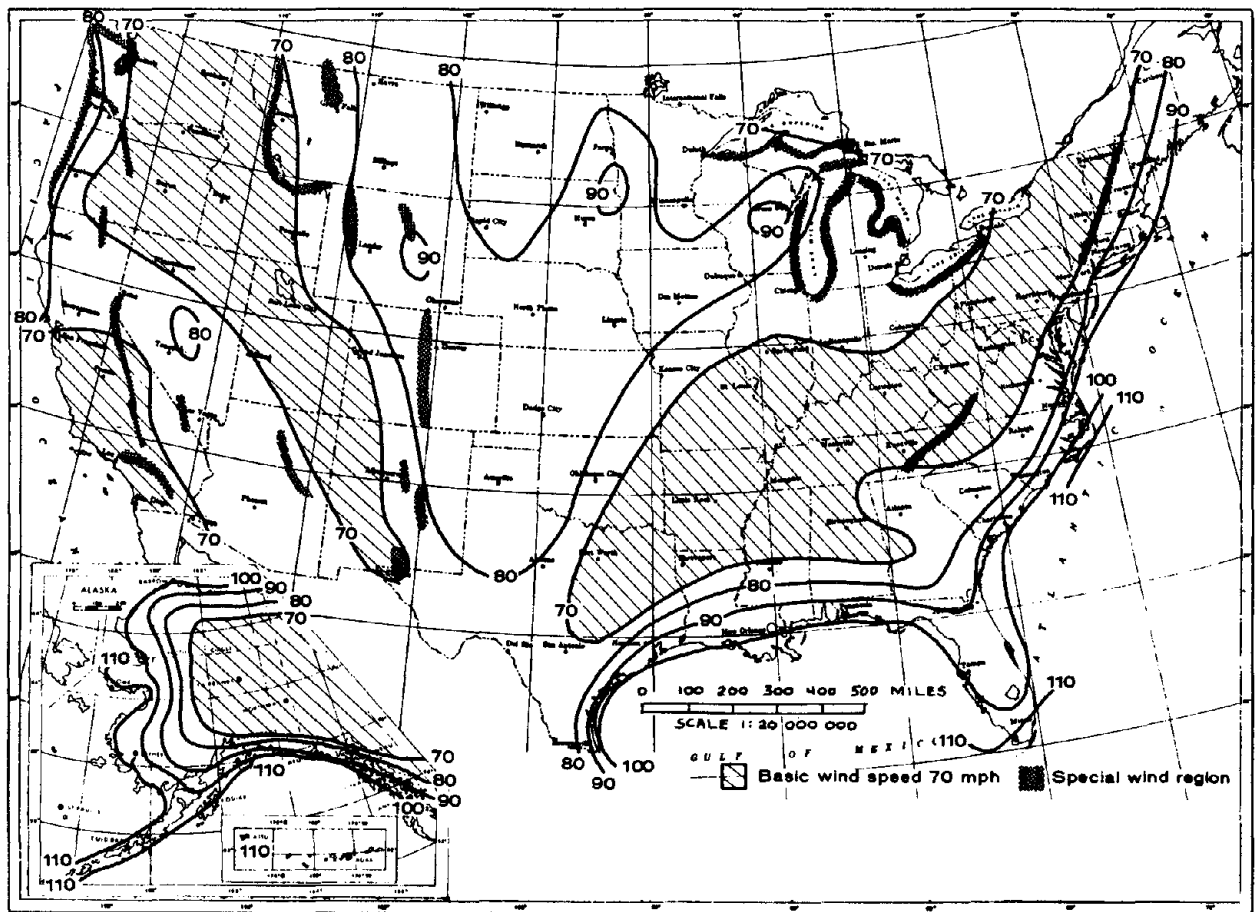
Notes:

(a)	Values are fastest-mile speeds at 33 ft (10 m) above ground for exposure category C and are associated with an annual probability of 0.02.
(b)	Linear interpolation between wind speed contours is acceptable.
(c)	Caution in the use of wind speed contours in mountainous regions of Alaska is advised.
(d)	Conversion: 1 mph = 1.609 km/hr

Figure 1112.3.2 - Basic Wind Speed (miles per hour)

APPENDIX D

FOUNDATION INVESTIGATION AND DESIGN

D.1   Subsurface Investigation

Since all the structural loads of the falsework are eventually transferred to the soil or rock underlying the falsework foundations, a subsurface investigation is necessary to determine the load-supporting ability of the various strata of soil or rock. Before designing the falsework foundations, it is necessary to determine the type, depth, and properties of the various soil or rock formations. If a foundation investigation has already been completed for the permanent structure, this information can be used for the design of the falsework foundations.

In the absence of site-specific information, additional soil exploration is recommended. The subsurface exploration can be tailored for each project and could include any of the following methods:

A.   The best information is generally obtained from soil borings completed throughout the area of the falsework foundations. The borings should be extended in depth to a level where the induced foundation pressures from the new loading will be less than 10 percent of the overburden pressure, but not less than 15 ft (5 m). Soil samples should be obtained at intervals not exceeding 2½ ft (0.8 m). The sampling should be done by conventional methods, resulting in test samples that are indicative of the strength and compressibility of the soil deposits. This should include standard penetration resistance testing (AASHTO T206-87), briefly described in Section D.3. In situ tests provide sufficient information for foundation design even though test samples are not obtained. These tests include cone penetrometer testing (ASTM D3441-86), pressuremeter testing (ASTM D4719-87), and dilatometer testing. Samples obtained by Shelby tubes (AASHTO T207-87) in cohesive soils are also acceptable when accompanied by appropriate laboratory tests, including the unconfined compressive strength test (AASHTO T208-90) and water content testing (AASHTO T265-86). Disturbed samples from auger cuttings are not considered acceptable.

B.   Test pits can be dug throughout the area to investigate the various soil or rock formations. Test pits should be used to supplement the soil-boring investigation wherever erratic or discontinuous subsurface conditions are present. It is easier to determine the thickness and character of these deposits from a large excavation than from examination of small diameter samples. The person logging the test pits should not only be capable of identifying the various strata, but should have some means for determining the relative density of each deposit. The hand penetrometer is sufficient for determining the shear strength of cohesive soils. Section D.4 provides guidelines for estimating the unconfined compressive strength of cohesive soils based on field observations. The determination of the relative density of granular soils is more difficult and may be subjective on the part of the observer. However, techniques such as the dynamic cone penetrometer can be used for this purpose. Alternatively, field density tests can be performed within the granular formations (AASHTO T191-86).

C.   Field tests can be undertaken and used as a guide where either the borings or test pits indicate questionable surface support conditions. This should include proof-testing of the ground surface with a fully loaded dump truck that has a minimum weight of 20 tons (18,000 kg). As the dump truck traverses the area, the amount of ground deflection under loading should be observed. Deflections of 2 in. (50 mm) or less under wheel load traffic are indicative of reasonably good support conditions. Large deflections and severe rutting are indicative of very poor support conditions.

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Plate load tests can be performed within the potential bearing strata for the foundations. The plate load test consists of a plate with a minimum 12-in. (305-mm) diameter with a jack used to provide a force and a truck or other heavy object used as a reaction. Deflections should be measured with either survey instruments or dial gauges. As the jack loads are applied, deflection readings should be taken at the design load and at twice the design load. The plate load test only measures the subgrade reaction of the soil within a depth of 1½ times the diameter of the plate. Other deeper strata could also affect future performance of the foundation.

D.2   Relative Density of Granular Deposits

In the United States, the most commonly used method of determining relative density of granular deposits is the standard penetration test (SPT). It is made by dropping a 140-lb (63.5-kg) hammer onto the drill rods from a height of 30 in. (0.76 m). The number of blows required to advance a split barrel sampling tube 1 ft (0.30 m) is called the standard penetration resistance. AASHTO T206-87 describes the test procedure. Table D.1 relates the SPT to relative density.

Table D.1   Determination of Relative Density Based on Standard Penetration Resistance

Standard Penetration Resistance	Relative Density
0-4	Very loose
4-10	Loose
10-30	Medium
30-50	Dense
Over 50	Very dense

Note: The standard penetration resistance (SPT) values correspond to an effective overburden pressure of 1 tsf (96 kN/m²). The correction factor, C_N, to be applied to field SPT values for other pressures is given by:

$$C_N = 0.77 \log_{10} \frac{20}{\bar{p}}$$ (D-1)

where $\bar{p}$ is the effective vertical overburden pressure in tsf at the elevation of the SPT test. This equation is valid for $\bar{p} \geq 0.25$ tsf (24 kN/m²).

In certain areas of the country, the cone penetration test (CPT) is used for soil exploration. The approximate relationship between SPT and CPT is shown in Table D.2.

Table D.2   CPT and SPT Values for Various Soils

Soil Type	$\frac{CPT}{SPT}$
Silts, sandy silts, slightly cohesive silt-sand mixtures	2
Clean fine to medium sands and slightly silty sands	3 to 4
Coarse sands and sands with little gravel	5 to 6
Sandy gravels and gravels	8 to 10

D.3   Consistency of Cohesive Soils

The most important index property used to describe cohesive soils is "consistency," which is qualitatively described as "soft," "medium," "stiff," or "hard." The consistency of a cohesive soil can also be quantitatively in terms of unconfined compressive strength. Table D.3 relates the qualitative terms for consistency to the quantitative values of unconfined compressive strength.

Table D.3 Consistency of Cohesive Soils[21]

Consistency	Field Identification	Unconfined Compressive Strength (tsf)
Very soft	Easily penetrated several inches by fist	Less than 0.25
Soft	Easily penetrated several inches by thumb	0.25 - 0.5
Medium	Can be penetrated several inches by thumb with moderate effort	0.5 - 1.0
Stiff	Readily indented by thumb but penetrated only with great effort	1.0 - 2.0
Very stiff	Readily indented by thumbnail	2.0 - 4.0
Hard	Indented with difficulty by thumbnail	Over 4.0

Notes:
(a)   ASTM D2488 has a slightly different criteria for describing consistency.
(b)   Conversion: 1 tsf = 96 kN/m^2

D.4   Unified Soil Classification System

The soil classification system most widely used by foundation engineers in the United States is known as the Unified Soil Classification System and has been adopted by the American Society for Testing and Materials as a Standard Test Method for Classification of Soils for Engineering Purposes, ASTM D-2487. The main points of ASTM D-2487 are summarized in Table D.4.

According to the Unified System, soils are categorized by particle-size and plasticity characteristics. Because this system is based on properties of the grains and of the remolded material, it does not fully describe the engineering properties of the intact material as encountered in the field. However, it permits reliable classification without extensive testing, and provides useful information about soils that have been properly classified by this method.

D.5   Potential Problem Soils

Certain soil deposits may experience movements that are not related to loading. These movements are due to the geologic composition of the deposits or climatic effects. The foundation designer should be cognizant of local problem soils and incorporate measures to reduce soil deformations.

In portions of the western and southern United States, collapsible soil deposits are sometimes encountered. These are generally fine sands and silts that are deposited in a very loose condition. A low amount of cementation holds the soil particles in the loose condition until they become wetted. Following wetting, the soils collapse into a denser state, thereby leading to significant settlements.

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Table D.4

Soil Classification according to the Unified Soil Classification System (ASTM D-2487).[21]

Major Divisions			Group Symbols	Typical Names	Classification Criteria
Coarse-Grained Soils — More than 50% retained on No. 200 sieve	Gravels — 50% or more of coarse fraction retained on No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines	$C_u = D_{60}/D_{10}$ Greater than 4 $C_c = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines	Not meeting both criteria for GW
		Gravels with Fines	GM	Silty gravels, gravel-sand-silt mixtures	Atterberg limits plot below "A" line or plasticity index less than 4
			GC	Clayey gravels, gravel-sand-clay mixtures	Atterberg limits plot above "A" line and plasticity index greater than 7
	Sands — More than 50% of coarse fraction passes No. 4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines	$C_u = D_{60}/D_{10}$ Greater than 6 $C_c = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			SP	Poorly graded sands and gravelly sands, little or no fines	Not meeting both criteria for SW
		Sands with Fines	SM	Silty sands, sand-silt mixtures	Atterberg limits plot below "A" line or plasticity index less than 4
			SC	Clayey sands, sand-clay mixtures	Atterberg limits plot above "A" line and plasticity index greater than 7
Fine-Grained Soils — 50% or more passes No. 200 sieve	Silts and Clays — Liquid limit 50% or less		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands	Plasticity chart
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
			OL	Organic silts and organic silty clays of low plasticity	
	Silts and Clays — Liquid limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts	
			CH	Inorganic clays of high plasticity, fat clays	
			OH	Organic clays of medium to high plasticity	
Highly organic soils			Pt	Peat, muck and other highly organic soils	Visual-manual identification

Classification on basis of percentage of fines: Less than 5% Pass No. 200 sieve: GW, GP, SW, SP. More than 12% Pass No. 200 sieve: GM, GC, SM, SC. 5% to 12% Pass No. 200 sieve: Borderline classification requiring use of dual symbols.

Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols.

Plasticity chart for classification of fine-grained soils and fine fraction of coarse-grained soils. Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols. Equation of A-line: $PI = 0.73\,(LL - 20)$

Throughout many parts of the United States, active soil deposits are present that can either significantly swell upon wetting or shrink upon drying. Shallow foundations supported upon these deposits should either be located below the limit of the moisture content variations, or the area around the foundation should be protected from an increase or decrease in moisture content.

In the northern portions of the United States where cold weather prevails, freezing of the bearing strata can occur and cause movements of falsework foundations. Sandy and silty soils are more susceptible to frost than other deposits, so foundations bearing upon these materials in cold weather shall be protected against frost penetrating into the ground. Protection can be afforded by a cover of fill or insulation such as styrofoam.

Fill deposits can vary widely in foundation support. If the fill was properly compacted in place, the support can be excellent, similar to a stiff clay or a medium dense sand. On the other hand, loosely dumped fill without compaction can be very compressible, similar to deposits consisting of a soft clay or loose sand. If not properly controlled, the fill can also be variable, with some weak pockets contained within a generally firm soil mass. This is especially true for cohesive soil deposits, where the water content at the time of placement is crucial to the degree of compaction that is achieved. For cohesive fill deposits, a thorough exploration program is necessary not only to determine the properties of the cohesive deposits, but also the uniformity of the entire fill mass.

D.6   Extended Foundations

For all deposits consisting of peat, organic silts, or very soft clays, and for some deposits with potential problem soils, such as collapsible or frost-susceptible soils, extended foundations will be necessary to carry the loads through the weak or problem deposits to more suitable foundation-bearing material. These extended foundations shall consist of very deep footings, drilled piers, or piles. Site improvement of organic soils and very soft clays by compaction or other means is not considered practical.

Where deep foundations will be used, the subsurface exploration shall extend to a depth of at least 10 ft (3 m) below the planned base of the foundation (unless rock is encountered at shallower depth) so that side friction and end-bearing capacity of the extended foundation can be calculated. The deep foundation design shall consider lateral as well as vertical loads.

If piles are to be used, the driving criteria shall be selected on the basis of a wave equation analysis or an accepted driving formula. The penetration data of the last 5 ft (1.5 m) of driving shall be recorded and submitted to the designer of the falsework foundations. Guidelines for the design of pile foundations are presented in chapter 4 of *AASHTO Standard Specifications for Highway Bridges.*[3]

If drilled piers are used, the base of each of the drilled piers shall be monitored to see that the proper bearing stratum has been reached and that the base of the drilled pier excavation is clean and free of sloughing and water prior to being filled with concrete. The strength of the bearing stratum shall be tested by the monitoring person and the results submitted to the designer of the falsework foundations. Guidelines for the design of drilled piers are presented in the American Concrete Institute publication *Suggested Design and Construction Procedures for Pier Foundations,* ACI 336.3R-72.[46]

D.7   AASHTO and ASTM Reference Standards

Standard Method of Test for Density of Soil In-Place by the Sand-Cone Method, (AASHTO T191)
Standard Method for Penetration Test and Split-Barrel Sampling of Soils, (AASHTO T206)
Standard Method for Thin-Walled Tube Sampling of Soils, (AASHTO T207)
Standard Method of Test for Unconfined Compressive Strength of Cohesive Soil, (AASHTO T208)
Standard Method of Test for Laboratory Determination of Moisture Content of Soils, (AASHTO T265)
Method of Deep, Quasi-Static, Cone and Friction-Cone Penetration Tests of Soil, (ASTM D-3441)
Test Method for Pressuremeter Testing in Soils, (ASTM D-4719)

APPENDIX E

CONVERSION OF EQUATIONS FROM U.S. CUSTOMARY UNITS TO S.I. METRIC UNITS

U.S. CUSTOMARY	METRIC

Section 2.1.2.2

Tension, axial, and flexural . . . . 22,000 psi

Tension, axial, and flexural . . . . . 152,000 N/mm^2

Compression, axial . . . . . . 16,000 - 0.38(L/r)^2 psi, except L/r shall not exceed 120.

Compression, axial . . . 110,000 - 2.62(L/r)^2 N/mm^2 except L/r shall not exceed 120.

Shear on gross section of web of rolled shapes . . . . 14,500 psi

Shear on gross section of web of rolled shapes . . . . 100,000 N/mm^2

Web crippling for rolled shapes . . . . . . 16,000 psi

Web crippling for rolled shapes . . 110,000 N/mm^2

Compression, flexural . . . . 12,000,000/(Ld/bt) psi, but not more than 22,000 psi.

Compression, flexural . . 83,000,000/(Ld/bt)N/mm^2 but not more than 152,000 N/mm^2

Section 2.2.5.2

$P_w = Kv^2$ (2-1)
P_w in psf;
v in fps.

$P_w = 515 Kv^2$
P_w in N/m^2;
v in m/s.

Section 3.2.2

$p = (w)(h)$ (3-1)
p in psf;
w in pcf;
h in ft.

$p = (w)(h)$
p in kN/m^2;
w in kN/m^3;
h in m.

Section 3.2.2.2

For columns:

$p = 150 + 9,000\ R/T$ (3-2)

For columns:

$p = 7,200 + 1,414,000\ R/(1.8T + 32)$

with a maximum of 3,000 psf, a minimum of 600 psf, but in no case greater than 150 h.

with a maximum of 144,000 N/m^2, a minimum of 29,000 N/m^2, but in no case greater than 23,600 h.

For walls with a rate of placement less than 7 ft per hour:

For walls with a rate of placement less than 2.1 m per hour:

$p = 150 + 9,000\ R/T$ (3-3)

$p = 7,200 + 1,414,000\ R/(1.8T + 32)$

with a maximum of 2000 psf, a minimum of 600 psf, but in no case greater than 150 h.

with a maximum of 96,000 N/m^2, a minimum of 29,000 N/m^2, but in no case greater than 23,600 h.

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U.S. CUSTOMARY	METRIC

For walls with a rate of placement of 7 to 10 ft per hour:

$$p = 150 + 43,400/T + 2,800R/T \qquad (3\text{-}4)$$

with a maximum of 2,000 psf, a minimum of 600 psf, but in no case greater than 150 h.
p in psf;
R in ft/hr;
T in °F.

For walls with a rate of placement of 2.1 to 3.0 m per hour:

$$p = 7,200 + (2,078,000 + 440,000R)/(1.8T + 32)$$

with a maximum of 96,000 N/m², a minimum of 29,000 N/m², but in no case greater than 23,600 h.
p in N/m²;
R in m/hr;
T in °C.

Section 4.3.1.2

$$P_a = K_a \gamma H \qquad (4\text{-}1)$$

P_a in psf;
γ in pcf;
H in ft.

$$P_a = K_a \gamma H$$

P_a in kN/m²;
γ in kN/m³;
H in m.

$$P_a = \gamma H - 2C \qquad (4\text{-}2)$$

P_a in psf;
γ in pcf;
H in ft;
C in psf;
Q_u in psf.

$$P_a = \gamma H - 2C$$

P_a in kN/m²
γ in kN/m³
H in m;
C in kN/m²;
Q_u in kN/m².

$$P_a = K_a \gamma H - 2C\sqrt{K_a} \qquad (4\text{-}3)$$

P_a in psf;
γ in pcf;
H in ft;
C in psf;
Q_u in psf.

$$P_a = K_a \gamma H - 2C\sqrt{K_a}$$

P_a in kN/m²;
γ in kN/m³;
H in m;
C in kN/m²;
Q_u in kN/m².

Section 4.3.1.3

$$P_o = K_o \gamma H \qquad (4\text{-}4)$$

P_o in psf;
γ in pcf;
H in ft.

$$P_o = K_o \gamma H$$

P_o in kN/m²;
γ in kN/m³;
H in m.

Section 4.3.1.4

$$P_p = K_p \gamma H \qquad (4\text{-}5)$$

P_p in psf;
γ in pcf;
H in ft.

$$P_p = K_p \gamma H$$

P_p in kN/m²;
γ in kN/m³;
H in m.

$$P_p = \gamma H + 2C \qquad (4\text{-}6)$$

P_p in psf;
γ in pcf;
H in ft;
C in psf.

$$P_p = \gamma H + 2C$$

P_p in kN/m²;
γ in kN/m³;
H in m;
C in kN/m².

$$P_p = K_p\gamma H + 2C\sqrt{K_p} \qquad (4\text{-}7)$$

P_p in psf;
γ in pcf;
H in ft;
C in psf.

$$P_w = Kv^2 \qquad (4\text{-}8)$$

P_w in psf;
v in fps.

Appendix B, Section 2

$$0.4\sqrt{\frac{P_{bf}}{F_{yc}}} \qquad (K1\text{-}1)$$

F_{yc} in ksi;
P_{bf} in kips;
t_f in in.

$$\frac{R}{t_w(N + 5k)} \le 0.66F_y \qquad (K1\text{-}2)$$

R in kips;
t_w in in.;
N in in.;
k in in.;
F_y in ksi.

$$\frac{R}{t_w(N + 2.5k)} \le 0.66F_y \qquad (K1\text{-}3)$$

R in kips;
t_w in in.;
N in in.;
k in in.;
F_y in ksi.

Appendix B, Section 4

$$R = 67.5t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{F_{yw}t_f/t_w} \qquad (K1\text{-}4)$$

R in kips;
t_w in in.;
N in in.;
d in in.;
t_f in in.;
F_{yw} in ksi.

$$P_p = K_p\gamma H + 2C\sqrt{K_p}$$

P_p in kN/m^2;
γ in kN/m^3;
H in m;
C in kN/m^2.

$$P_w = 515\ Kv^2$$

P_w in N/m^2;
v in m/s.

$$0.4\sqrt{\frac{P_{bf}}{F_{yc}}}$$

F_{yc} in N/mm^2;
P_{bf} in N;
t_f in mm.

$$\frac{R}{t_w(N + 5k)} \le 0.66F_y$$

R in N;
t_w in mm;
N in mm;
h in mm;
F_y in N/mm^2.

$$\frac{R}{t_w(N + 2.5k)} \le 0.66F_y$$

R in N;
t_w in mm;
N in mm;
k in mm;
F_y in N/mm^2.

$$R = 177t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{F_{yw}t_f/t_w}$$

R in N;
t_w in mm;
N in mm;
d in mm;
t_f in mm;
F_{yw} in N/mm^2.

$$R = 34t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{F_{yw}t_f/t_w} \qquad \text{(K1-5)}$$

R in kips;
t_w in in.;
N in in.;
d in in.;
t_f in in.;
F_{yw} in ksi.

$$R = 89t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{F_{yw}t_f/t_w}$$

R in N;
t_w in mm;
N in mm;
d in mm;
t_f in mm;
F_{yw} in N/mm^2.

Appendix B, Section 5

$$R = \frac{6{,}800t_w^3}{h}\left[1 + 0.4\left(\frac{d_c/t_w}{l/b_f}\right)^3\right] \qquad \text{(K1-6)}$$

R in kips;
t_w in in.;
h in in.;
d_c in in.;
l in in.;
b_f in in.

$$R = \frac{46{,}900t_w^3}{h}\left[1 + 0.4\left(\frac{d_c/t_w}{l/b_f}\right)^3\right]$$

R in N;
t_w in mm;
h in mm;
d_c in mm;
l in mm;
b_f in mm.

$$R = \frac{6{,}800t_w^3}{h}\left[0.4\left(\frac{d_c/t_w}{l/b_f}\right)^3\right] \qquad \text{(K1-7)}$$

R in kips;
t_w in in.;
h in in.;
d_c in in.;
l in in.;
b_f in in.

$$R = \frac{46{,}900t_w^3}{h}\left[0.4\left(\frac{d_c/t_w}{l/b_f}\right)^3\right]$$

R in N;
t_w in mm;
h in mm;
d_c in mm;
l in mm;
b_f in mm.

Appendix B, Section 6

$$\frac{4{,}100t_{wc}^3\sqrt{F_{yc}}}{P_{bf}} \qquad \text{(K1-8)}$$

d_c in in.;
t_{wc} in in.;
F_{yc} in ksi;
P_{bf} in kips.

$$\frac{10{,}800t_{wc}^3\sqrt{F_{yc}}}{P_{bf}}$$

d_c in mm;
t_{wc} in mm;
F_{yc} in N/mm^2;
P_{bf} in N.

U.S. CUSTOMARY	METRIC

Appendix B, Section 8

$$A_{st} = \frac{P_{bf} - F_{yc}t_{wc}(t_b + 5k)}{F_{yst}} \quad \text{(K1-9)}$$

A_{st} in in.^{2};
P_{bf} in kips;
F_{yc} in ksi;
t_{wc} in in.;
t_b in in.;
k in in.;
F_{yst} in ksi.

$$A_{st} = \frac{P_{bf} - F_{yt}t_{wc}(t_b + 5k)}{F_{yst}}$$

A_{st} in mm^2;
P_{bf} in N;
F_{yc} in N/mm^2;
t_{wc} in mm;
t_b in mm;
k in mm;
F_{yst} in N/mm^2.

Appendix C, Section 2316

$$P = C_e C_q q_s I \quad \text{(16-1)}$$
P in psf;
q_s in psf.

$$P = C_e C_q q_s I$$
P in N/m^2;
q_s in N/m^2.

Appendix C, Section 6.5.2

$$F = q_z G_h C_f A_f$$
F in lb;
q_z in psf;
A_f in ft^2.

$$F = q_z G_h C_f A_f$$
F in N;
q_z in N/m^2;
A_f in m^2.

$$q_z = 0.00256\, K_z\,(IV)^2$$
q_s in psf;
v in mph.

$$q_s = 0.0473\, K_z\,(IV)^2$$
q_s in N/m^2;
V in km/hr.

Appendix C, Section 1112.3

$$P_d = P_e I^2 C_p$$
P_d in psf;
P_e in psf.

$$P_d = P_e I^2 C_p$$
P_d in N/m^2;
P_e in N/m^2.

Appendix D, Section D.2

$$C_n = 0.77 \log_{10} \frac{20}{\bar{p}} \quad \text{(D-1)}$$
$\bar{p}$ in tsf.

$$C_n = 0.77 \log_{10} \frac{1920}{\bar{p}}$$
$\bar{p}$ in kN/m^2.

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