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16. Abstract This report synthesizes the results of Reports 1126-1, 1126-2, and 1126-3 in the form of a strength-based design guide for steel-to-concrete connections involving short anchor bolts. Both single- and multiple-anchor behavior are addressed. This <i>Design Guide</i> identifies the basic elements of any structural connection to concrete, specifies the desired behavior for each element, and describes the design procedures to be followed to produce that desired behavior. The <i>Design Guide</i> addresses the baseplate, the anchor itself, and the embedment of the anchor in the surrounding concrete. The <i>Design Guide</i> is intended to apply to cast-in-place anchors, grouted anchors, torque-controlled expansion anchors, undercut anchors, and adhesive anchors. The <i>Design Guide</i> is oriented primarily towards anchors that are ductile in that their failure is governed by yield and fracture of the anchor steel. However, sections of the guide deal with the design of non-ductile anchors. The <i>Design Guide</i> does not specify nominal or allowable load values for individual anchor types. However, it does describe in detail procedures which are recommended for use by the Highway Department, by testing laboratories, or by the manufacturers themselves to arrive at such values. It also describes in detail procedures which are recommended for use by the Highway Department in evaluating the installation of anchor bolts.					
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DESIGN GUIDE FOR STEEL-TO-CONCRETE CONNECTIONS

by

R. A. COOK, G. T. DOERR, AND R. E. KLINGNER

**Research Report No. 1126-4F
Research Project 3-5-83-1126
“Design Guide for Short Anchor Bolts”**

**Conducted for
Texas
State Department of Highways and Public Transportation
In Cooperation with the
U.S. Department of Transportation
Federal Highway Administration**

by

**CENTER FOR TRANSPORTATION RESEARCH
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There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design or composition of matter, or any new and useful improvement thereof, or any variety of plant which is or may be patentable under the patent laws of the United States of America or any foreign country.

PREFACE

Many structural details in current use by the Texas State Department of Highways and Public Transportation (SDHPT) involve the use of anchor bolts, sometimes in retrofit applications. Examples are attachment of traffic barriers to structures, attachment of bridge girders to bearing blocks, attachment of end fixtures to precast concrete components, and attachment of steel members to existing concrete. Anchors are of different types: cast-in-place, grouted, adhesive, expansion, or undercut. These anchors are now designed using procedures which are outdated and often erroneous. Recent investigations have suggested that various Texas SDHPT designs involving anchor bolts are inconsistent and possibly unconservative.

This report synthesizes the results of Texas SDHPT Project 1126, "Design Guide for Short Anchor Bolts." The purpose of Project 1126 was to improve existing design procedures for cast-in-place anchor bolts, and to develop rational and dependable procedures for retrofit installation of anchor bolts in the form of an easy-to-use design guide. This report is that guide. In it, design procedures are recommended for all aspects of connections to concrete involving single or multiple anchors.

SUMMARY

This report synthesizes the results of Reports 1126-1, 1126-2, and 1126-3 in the form of a strength-based design guide for steel-to-concrete connections involving short anchor bolts. Both single- and multiple-anchor behavior are addressed. This *Design Guide* identifies the basic elements of any structural connection to concrete, specifies the desired behavior for each element, and describes the design procedures to be followed to produce that desired behavior. The *Design Guide* addresses the baseplate, the anchor itself, and the embedment of the anchor in the surrounding concrete. The *Design Guide* is intended to apply to cast-in-place anchors, grouted anchors, torque-controlled expansion anchors, undercut anchors, and adhesive anchors.

The *Design Guide* is oriented primarily towards anchors that are ductile in that their failure is governed by yield and fracture of the anchor steel. However, sections of the guide deal with the design of non-ductile anchors.

The *Design Guide* does not specify nominal or allowable load values for individual anchor types. However, it does describe in detail procedures which are recommended for use by the Highway Department, by testing laboratories, or by the manufacturers themselves to arrive at such values. It also describes in detail procedures which are recommended for use by the Highway Department in evaluating the installation of anchor bolts.

IMPLEMENTATION

This *Design Guide* addresses all aspects of steel-to-concrete connections to concrete involving short anchor bolts. It contains recommended procedures for determining nominal or allowable load values for different types of anchors, and for evaluating the installation of anchors. The *Design Guide* should be used by the Texas SDHPT for design, qualification, and evaluation of steel-to-concrete connections involving short anchor bolts.

TABLE OF CONTENTS

	<u>Page</u>
1. GENERAL	1
1.1 Scope	1
1.2 General Design Philosophy	1
1.3 General Design Requirements	3
1.4 Strength Reduction Factors	3
1.4.1 Factored Loads	3
1.4.2 Unfactored Loads	3
1.5 Design Procedure	4
1.6 Definitions	5
1.7 Notation	6
2. SINGLE ANCHORS: GENERAL CONSIDERATIONS FOR DESIGN	11
2.1 Types of Anchors	11
2.2 Failure Mechanisms	11
2.3 Nominal Strength of Steel for Single Anchors	11
2.3.1 Nominal Tensile Strength of Steel for Single Anchors	11
2.3.2 Nominal Shear Strength of Steel for Single Anchors	11
2.3.3 Nominal Strength of Steel for Single Anchors in Tension and Shear	11
2.4 Nominal Strength of Embedment for Single Anchors	14
2.4.1 Nominal Tensile Strength of Embedment for Single Cast- in-place Anchors	14
2.4.2 Nominal Tensile Strength of Embedment for Single Undercut Anchors	17
2.4.3 Nominal Tensile Strength of Embedment for Single Adhesive Anchors	17
2.4.4 Nominal Tensile Strength of Embedment for Single Grouted Anchors	20
2.4.5 Nominal Tensile Strength of Embedment for Single Expansion Anchors	20
2.4.6 Nominal Shear Strength of Embedment for Single Anchors	22
2.4.7 Nominal Strength of Embedment for Single Anchors in Tension and Shear	24
2.5 Ductile and Non-Ductile Anchors	24
2.6 Special Case: Quasi-Ductile Anchors	24
2.7 Anchor Head Requirements	26

3.	STEEL DESIGN FOR DUCTILE CONNECTIONS	27
3.1	General Requirements	27
3.2	Design Philosophy	27
3.3	Design Procedure for Moment	31
3.4	Design Procedure for Moment with Axial Compression	33
3.5	Design Procedure for Moment with Axial Tension	33
3.6	Design Procedure for Shear	33
4.	EMBEDMENT REQUIREMENTS FOR DUCTILE CONNECTIONS	35
4.1	General Requirements	35
4.2	Design Philosophy	35
4.3	Nominal Strength of Steel for Ductile Connection Embedment Design	35
4.3.1	Nominal Tensile Strength of Steel for Ductile Connection Embedment Design	35
4.3.2	Nominal Shear Strength of Steel for Ductile Connection Embedment Design	35
4.4	Nominal Strength of Embedment for Connections	36
4.4.1	Nominal Tensile Strength of Embedment for Connections with Cast-in-place Tension-Anchors	36
4.4.2	Nominal Tensile Strength of Embedment for Connections with Undercut Tension-Anchors	39
4.4.3	Nominal Tensile Strength of Embedment for Connections with Adhesive Tension-Anchors	39
4.4.4	Nominal Tensile Strength of Embedment for Connections with Grouted Tension-Anchors	39
4.4.5	Nominal Tensile Strength of Embedment for Connections with Expansion Tension-Anchors	39
4.4.6	Nominal Shear Strength of Embedment for Connections with Shear-Anchors	39
4.5	Calculation of Projected Area for Overlapping Failure Cones	40
5.	DESIGN OF NON-DUCTILE CONNECTIONS	43
5.1	General Requirements	43
5.2	Design Philosophy	43

6.	DESIGN OF BASEPLATES	45
6.1	General Requirements	45
6.2	Design Philosophy	45
6.3	Baseplate Design Procedure for Flexure	47
6.4	Baseplate Design Procedure for Shear	47
6.5	Baseplate Design Procedure for Concrete Bearing	48
6.6	Baseplate Design Procedure for Steel Bearing	48
6.7	Baseplate Anchor Hole Requirements	48
	APPENDIX A - INSTALLATION REQUIREMENTS FOR RETROFIT ANCHORS	49
	APPENDIX B - INSPECTION AND TESTING REQUIREMENTS FOR RETROFIT ANCHORS	53
	REFERENCES	57

LIST OF FIGURES

<u>Figure</u>	<u>Page</u>
1.1 Typical Steel-to-Concrete Connection	2
1.2 Type of Anchors	2
2.1 Anchor Yielded by Tension	12
2.2 Anchor Yielded by Shear	12
2.3 Anchor Yielded by Combined Tension and Shear	13
2.4 Interaction Equations for Combined Tension and Shear	13
2.5 Pullout-Core Failure	15
2.6 Blowout-Cone Failure	15
2.7 Design Pullout Cone for Cast-in-Place Anchors	16
2.8 Design Pullout Cone Limited by Free Edge	16
2.9 Design Blowout Cone	18
2.10 Design Pullout Cone for Undercut Anchors	18
2.11 Plug/Cone-Pullout Failure	19
2.12 Plug-Pullout Failure	19
2.13 Excessive Slip Failure with Shallow Pullout Cone	21
2.14 Pullout Cone Failure	21
2.15 Pushout Cone Failure	23
2.16 Design Pushout Cone	23
2.17 Quasi-Ductile Anchors	25
3.1 Assumed Location of Compressive Reaction for Moment	29
3.2 Alternate Location of Compressive Reaction for Moment	30
3.3 Forces Acting at the Steel/Concrete Interface	32
4.1 Projected Areas for Overlapping Cones	37
4.2 Projected Area as Limited by Concrete Thickness	38
4.3 Approximate Projected Areas for Overlapping Cones	41
6.1 Typical Deformed Shape of “Reasonably” Flexible Baseplate	46
B.1 Schematic Drawing of Loading System	54

LIST OF TABLES

<u>Table</u>		<u>Page</u>
1.1	Tensile Stress Area for Threaded Anchors	7

1. GENERAL

1.1 Scope

This design guide provides minimum requirements for the design of connections between steel members and concrete. Figure 1.1 shows a typical steel-to-concrete connection. Design methods are presented for steel-to-concrete connections using cast-in-place anchors, undercut anchors, adhesive anchors, grouted anchors, and expansion anchors. Figure 1.2 shows typical types of anchors.

The design provisions given in this *Guide* are based primarily on the references given in Appendix C. Appendix C contains a cross-reference for the Sections of the *Guide* and the references. Design provisions that are unique to this *Guide* are justified in the *Guide*.

1.2 General Design Philosophy

To determine the ultimate strength of a steel-to-concrete connection, two separate strengths must be considered:

- 1) The strength of the steel
- 2) The strength of the embedment

The lesser of these two strengths represents the ultimate strength of the connection. In the simple case of a single cast-in-place headed anchor loaded in tension, the strength of the steel is the tensile strength of the anchor itself, while the strength of the embedment is related to the embedded length of the anchor and the tensile strength of the concrete. If the anchor is embedded far enough into the concrete, the strength of the steel controls. Such a connection can be described as ductile.

For the purposes of this design guide, ductility is defined as the ability of a structural component to undergo significant inelastic deformation at predictable loads, and without significant loss of strength. A connection to concrete is ductile if its ultimate strength is controlled by the strength of the steel. A ductile connection to concrete fails by yielding and fracture of the anchors. A connection to concrete is non-ductile if its ultimate strength is controlled by the strength of the embedment. Non-ductile connections fail by brittle fracture of the concrete in tension, and by unpredictable failure modes which occur in the concrete, such as anchor slip without steel fracture.

The design methods of this guide are based on the following general design philosophy:

- 1) Ductile steel-to-concrete connections are preferred since load redistribution and energy absorption occur before failure. Insofar as possible, the ultimate strength of a connection should be controlled by the strength of the steel in the connection.
- 2) Non-ductile steel-to-concrete connections are acceptable provided that the design method is sufficiently conservative and does not assume any load redistribution among the anchors.

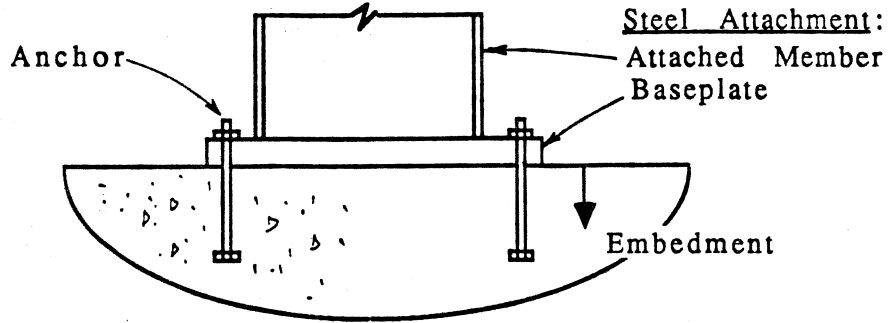
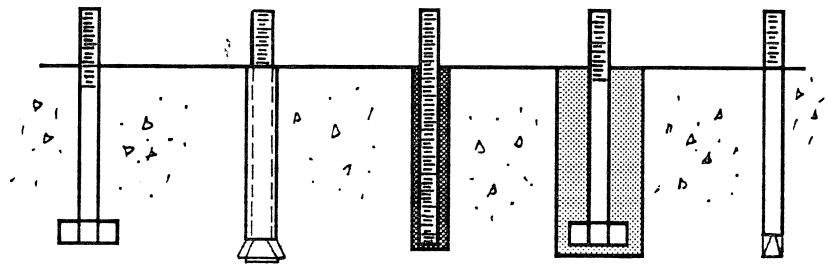


Fig. 1.1 Typical Steel-to-Concrete Connection



Cast-in-Place Undercut Adhesive Grouted Expansion

Fig. 1.2 Types of Anchors

1.3 General Design Requirements

- 1) The design strength of a connection shall be greater than or equal to the applied load.
- 2) The design strength of a connection shall be taken as the nominal strength calculated in accordance with the requirements and assumptions of this *Guide*, multiplied by a strength reduction factor ϕ .
- 3) The applied load may be either a factored load developed from an ultimate strength design (USD) approach, or an unfactored load developed from a working stress design (WSD) approach. Subsection 1.4.1 provides values for the strength reduction factor ϕ for cases involving factored loads. Subsection 1.4.2 provides values for the strength reduction factor ϕ for cases involving unfactored loads.
- 4) Anchor preload shall not be considered when using the provisions of this guide. Anchor preload does not affect the ultimate strength of a connection but does improve connection performance at working load levels. Anchor preload equal to 70% of the specified minimum tension strength of the anchor is recommended for improved performance at working load levels.

1.4 Strength Reduction Factors

1.4.1 Factored Loads: For cases involving factored loads (USD), the strength reduction factor ϕ shall be as follows:

Baseplate steel in flexure, based on F_y	$\phi_b = 0.90$
Baseplate steel in shear, based on F_y	$\phi_v = 0.90$
Baseplate steel in bearing, based on F_u	$\phi_p = 0.75$
Anchor steel in tension, based on F_u	$\phi_t = 0.75$
Anchor steel in shear, based on F_u	$\phi_v = 0.65$
Concrete in bearing	$\phi_p = 0.70$
Concrete in tension, friction, or bond (See Note in Subsection 2.4.3)	$\phi_e = 0.65$

The strength reduction factors for baseplate steel and anchor steel are consistent with the AISC LRFD Specifications [8]. The strength reduction factors for concrete in bearing and concrete in tension are consistent with ACI 349-85 [4]. The strength reduction factor for concrete in friction or bond is consistent with test results [2,3].

1.4.2 Unfactored Loads: For cases involving unfactored loads (WSD), the strength reduction factor ϕ shall be as follows:

Baseplate steel in flexure, based on F_y	$\phi_b = 0.50$
Baseplate steel in shear, based on F_y	$\phi_v = 0.67$
Baseplate steel in bearing, based on F_u	$\phi_p = 0.62$

Anchor steel in tension, based on F_u	$\phi_t = 0.44$
Anchor steel in shear, based on F_u	$\phi_v = 0.37$
Concrete in bearing	$\phi_p = 0.35$
Concrete in tension, friction, or bond (See Note in Subsection 2.4.3)	$\phi_e = 0.33$

The strength reduction factors for baseplate steel and anchor steel give results that are consistent with the provisions of the AISC Working-Stress Specifications [7] when used with the design procedures of this *Guide*. The strength reduction factors for concrete are taken as one-half of the corresponding values used for factored loads. This factor of one-half is consistent with the provisions of ACI 322-72 [5], which uses an unfactored load strength (WSD) equal to one-half of the factored load strength (USD) for concrete in bearing or tension.

1.5 Design Procedure

The design of steel-to-concrete connections is an iterative process. The following procedure is suggested:

- 1) Choose an anchor that is capable of ductile behavior. Cast-in-place, undercut, adhesive, grouted, and some expansion anchors are capable of ductile behavior. Refer to Section 2.5 for information on ductile and non-ductile anchors.

NOTE: If non-ductile anchors are used the design must meet the requirements of Section 5, and Steps 2 and 3 of this procedure are not applicable. Connections designed under Section 5 are non-ductile, inefficient, and more difficult to design. They should be avoided if possible.

- 2) Perform a preliminary design assuming the connection is a ductile connection. Determine the number, spacing, and size of anchors in accordance with Section 3.
- 3) Determine if the anchor pattern chosen can be embedded sufficiently to produce a ductile connection. Section 4 provides the embedment requirements for ductile connections.
 - a) If sufficient embedment can be provided to produce a ductile connection, the preliminary design is adequate.
 - b) If the preliminary design does not produce a ductile connection, try other patterns designed in accordance with Section 3.
 - c) If the connection cannot be embedded for ductile behavior (as defined in Section 4), the design procedures of Section 5 (non-ductile connections) must be used.
- 4) Design the baseplate in accordance with Section 6.

1.6 Definitions

- Adhesive Anchor - A retrofit anchor that consists of a threaded rod installed in a hole with a diameter about one-sixteenth to one-eighth inch larger than the diameter of the rod. The hole is filled with a structural adhesive (for example, an epoxy, vinylester, or polyester). Adhesive anchors transfer tensile load to the concrete by bond along the embedded length of the anchor.
- Anchor - The part of a steel-to-concrete connection that actually does the connecting. Anchors may be cast-in-place, undercut, adhesive, grouted, or expansion.
- Attached Member - The structural steel member being connected to the concrete.
- Baseplate - The steel plate that is connected to the attached member and is in contact with the concrete.
- Cast-in-Place Anchor - A headed anchor that is cast into the concrete and transfers tensile load to the concrete by bearing on the embedded head.
- Ductile Anchor - An anchor that can be sufficiently embedded so that failure will occur by yielding and fracture of the steel when loaded in direct tension. Cast-in-place anchors, undercut anchors, adhesive anchors, grouted anchors, and some expansion anchors can be ductile anchors if sufficiently embedded. The use of ductile anchors in a connection does not ensure a ductile connection, but does allow for the possibility of a ductile connection.
- Ductile Connection - A connection whose design strength is controlled by the strength of the steel.
- Ductility - The ability of a structural component to undergo significant inelastic deformation at predictable loads, without significant loss of strength.
- Embedment - The portion of a steel-to-concrete connection inside the concrete.
- Expansion Anchor - A retrofit anchor that transfers tension load to the concrete by friction between the anchor and the concrete at the base of a hole drilled in the concrete. The friction force results from a compressive reaction generated in opposition to the movement of an expansion mechanism at the embedded end of the anchor.
- Fully Bonded Anchor - An adhesive anchor that is bonded along the entire embedded length of the anchor.
- Grouted Anchor - A retrofit anchor that consists of a headed anchor installed in a hole with a diameter about one and one-half inches larger than the diameter of the anchor. The hole is filled with a non-shrink grout, usually containing portland cement, hydraulic cement, sand, and various chemicals to reduce shrinkage. Grouted anchors transfer tensile load

to the concrete by bearing on the anchor head, and by bond along the grout/concrete interface.

Headed Anchor - A headed bolt, a headed stud, a threaded stud with an end nut, or a threaded rod with an end nut.

Non-Ductile Anchor - An anchor that cannot be sufficiently embedded so that failure will occur by yielding and fracture of the steel when loaded in direct tension. Most expansion anchors are non-ductile anchors. A connection that uses non-ductile anchors cannot be a ductile connection.

Non-Ductile Connection - A connection whose design strength is controlled by the strength of the embedment.

Partially Bonded Anchor - An adhesive anchor that is intentionally debonded at the top portion of its embedded length (usually over a length of about 2 inches).

Quasi-Ductile Anchor - An anchor that is not embedded sufficiently to fail by yielding and fracture of the anchor, but that does fail by yielding and fracture of embedment reinforcing steel after an initial brittle failure of the concrete.

Retrofit Anchor - An anchor which is installed in hardened concrete.

Shear-Anchor - An anchor which is assumed to carry shear.

Steel Attachment - The attached member and its baseplate.

Tension-Anchor - An anchor which is assumed to carry tension.

Undercut Anchor - A retrofit anchor that transfers tensile load to the concrete by bearing of an expansion device against a bell-shaped enlargement of the hole at the base of the anchor. An undercut anchor consists of a sleeve, a threaded rod, and an expansion device.

1.7 Notation

A_1 = Loaded area

A_2 = Maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area

A_p = Projected area of assumed failure cones at the surface of the concrete for single anchors.

A_{pa} = Approximate projected area of assumed failure cones at the surface of the concrete for groups of anchors with overlapping failure cones as defined in Section 4.5

A_{pg} = Projected area of assumed failure cones at the surface of the concrete for groups of anchors with overlapping failure cones.

Table 1.1 Tensile Stress Area for Threaded Anchors

Anchor Diameter d_b (in)	Threads Per Inch n	Tensile Stress Area A_s (in ²)
1/4	20	0.032
3/8	16	0.078
1/2	13	0.142
5/8	11	0.226
3/4	10	0.334
7/8	9	0.462
1	8	0.606
1-1/8	7	0.763
1-1/4	7	0.969
1-3/8	6	1.16
1-1/2	6	1.41
1-3/4	5	1.90
2	4-1/2	2.50
2-1/4	4-1/2	3.25
2-1/2	4	4.00
2-3/4	4	4.93
3	4	5.97
3-1/4	4	7.10
3-1/2	4	8.33
3-3/4	4	9.66
4	4	11.10

A_s = Effective cross-sectional area of an anchor. The effective cross-sectional area shall be taken as the tensile stress area of threaded anchors or the full cross-sectional area of unthreaded anchors. The tensile stress area of threaded anchors shall be determined in accordance with American National Standards Institute Specification ANSI B1.1 as:

$$\text{Tensile stress area} = 0.7854 \left(\frac{d_b - 0.9743}{n} \right)^2$$

Table 1.1 shows the tensile stress areas for UNC series threaded anchors.

- A_t = Area of tension steel in a ductile connection
 A_{tc} = ΣA_s for all tension-anchors in a connection
 A_v = Area of shear steel in a ductile connection
 A_{vc} = ΣA_s for all shear-anchors in a connection
 b_f = width of the compression element of the attached member
 C = Compressive reaction from applied moment and axial load
 d = Internal moment arm for a ductile connection; distance between the compressive resultant and the centroid of the tensile steel
 d' = Distance between a tension-anchor and the nearest edge of the attached member
 d_b = Nominal diameter of an anchor
 d_h = Nominal diameter of the hole for adhesive and grouted anchors
 f'_c = Specified compressive strength of concrete, psi.
 $\sqrt{f'_c}$ = Square root of specified compressive strength of concrete, psi.
 F_u = Specified minimum tensile strength of steel
 F_{ut} = Average value for tensile strength of steel. F_{ut} shall be taken as 1.2 F_u if the actual value is not known. The 1.2 factor accounts for the average tensile strength being 20% larger than the specified minimum tensile strength.
 F_y = Specified minimum yield strength of steel
 ℓ_c = Assumed depth of failure cones for embedment design
 ℓ_d = Development length for embedment reinforcing steel
 ℓ_e = Embedded length of an anchor; distance from concrete surface to bearing surface of anchor head or expansion mechanism for cast-in-place, undercut, grouted, and expansion anchors; entire embedded length for adhesive anchors
 m = Distance from the centerline of an anchor to a free edge of the concrete
 m_p = Nominal flexural capacity per unit width, of a baseplate based on the plastic section modulus
 m_y = Nominal flexural capacity per unit width, of a baseplate based on the elastic section modulus
 m_{ut} = Maximum moment per unit width induced in a baseplate by the tension-anchors, based on F_{ut} of the anchors
 M = Applied moment
 M_n = Nominal moment capacity

- n = Number of threads per inch for threaded anchors, used in calculating the tensile stress area (Section 1); the number of anchors in a closely spaced group of anchors (Section 4); or the number of anchors in a row of tension-anchors (Section 6)
- P_n = Nominal bearing strength
- s = Anchor spacing in a row of anchors
- t = Thickness of baseplate
- T_e = Nominal tensile strength of the embedment for an isolated single anchor or for an individual anchor in a group of tension-anchors with overlapping failure cones
- T_{ec} = Nominal tensile strength of the embedment for the tension-anchors in a connection
- T_n = Nominal tensile strength of a single anchor in combined tension and shear
- T_s = Nominal tensile strength of the steel
- T_{sc} = Nominal tensile strength of the steel for the tension-anchors in a connection
- u_0 = Specified bond strength for adhesive or grouted anchors, ksi.
- v_n = Nominal shear strength per unit width, of a baseplate
- v_{ut} = Maximum shear per unit width induced in a baseplate by the tension-anchors, based on F_{ut} of the anchors
- V = Applied shear
- V_e = Nominal shear strength of the embedment for an isolated single anchor or for an individual anchor in a group of shear-anchors with overlapping failure cones
- V_{ec} = Nominal shear strength of the embedment for the shear-anchors in a connection
- V_n = Nominal shear strength of a single anchor in combined tension and shear (Section 2), or the nominal shear strength of a connection (Section 3 and Section 5)
- V_s = Nominal shear strength of the steel
- V_{sc} = Nominal shear strength of the steel for the shear-anchors in a connection
- V_μ = Nominal frictional shear strength
- α = Ratio of the largest anchor spacing between adjacent anchors in a group of anchors with overlapping failure cones to the radius of an individual anchor failure cone. Anchor spacing measured within rows of anchors.
- β = Reduction factor applied to the approximate projected area of overlapping failure cones as defined in Section 4.4
- γ = Ratio of the nominal shear strength of the steel to the nominal tensile strength of the steel for an anchor (taken equal to 0.60 for undercut anchors, 0.50 for others)
- λ' = Specified elastic property for adhesive of grouted anchors, ksi.

- $\mu =$ Coefficient of friction between steel and concrete (taken equal to 0.4)
- $\phi =$ Strength reduction factor. See Section 1.4
- $\Theta =$ Base angle for assumed failure cones

2. SINGLE ANCHORS: GENERAL CONSIDERATIONS FOR DESIGN

2.1 Types of Anchors

Anchors can be either cast-in-place or retrofit. A typical cast-in-place anchor is a headed bolt installed in position before the concrete is placed. Retrofit anchors are installed after the concrete has hardened, and can be either undercut, adhesive, grouted, or expansion.

2.2 Failure Mechanisms

The strength of a single anchor is controlled by the lesser of the strength of the steel or the strength of the embedment. Section 2.3 provides methods for calculating the nominal strength of the steel for single anchors. Section 2.4 provides methods for calculating the nominal strength of the embedment for single anchors.

2.3 Nominal Strength of Steel for Single Anchors

2.3.1 Nominal Tensile Strength of Steel for Single Anchors: The steel failure mechanism for anchors in tension is usually characterized by tensile yielding and fracture of the threaded portion of the anchor. Figure 2.1 shows the deformed shape of an anchor that has yielded in tension.

The nominal tensile strength of the steel for single anchors is given by:

$$T_s = A_s F_u \quad (2-1)$$

2.3.2 Nominal Shear Strength of Steel for Single Anchors: Anchors transfer shear by bearing of the baseplate against the anchor, and by bearing of the anchor against the concrete. The steel failure mechanism for anchors in shear is characterized by yielding and fracture of the threaded portion of the anchor due to kinking and bending of the anchor. Local crushing of the concrete occurs but does not limit the strength of the anchor. Figure 2.2 shows the deformed shape of an anchor that has yielded in shear.

The nominal shear strength of the steel for single anchors is given by:

$$V_s = \gamma T_s \quad (2-2)$$

2.3.3 Nominal Strength of Steel for Single Anchors in Tension and Shear: The steel failure mechanism for anchors in combined tension and shear is characterized by yielding and fracture in the threaded portion of the anchor due to tension, kinking, and bending. Figure 2.3 shows a typical deformed shape for an anchor in tension and shear.

The nominal strength of the steel for single anchors in tension T_n and shear V_n may be conservatively determined using a linear interaction based on T_s and V_s , as defined by Eq. (2-1) and Eq. (2-2):

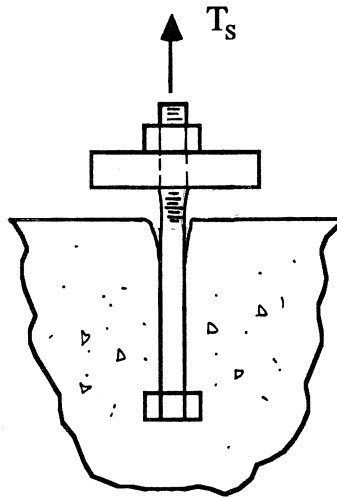


Fig. 2.1 Anchor Yielded by Tension

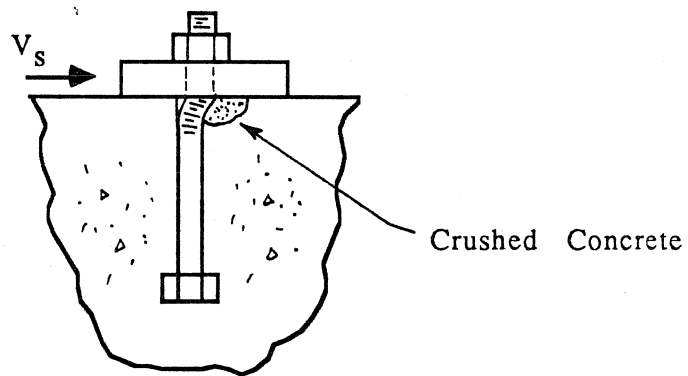


Fig. 2.2 Anchor Yielded by Shear

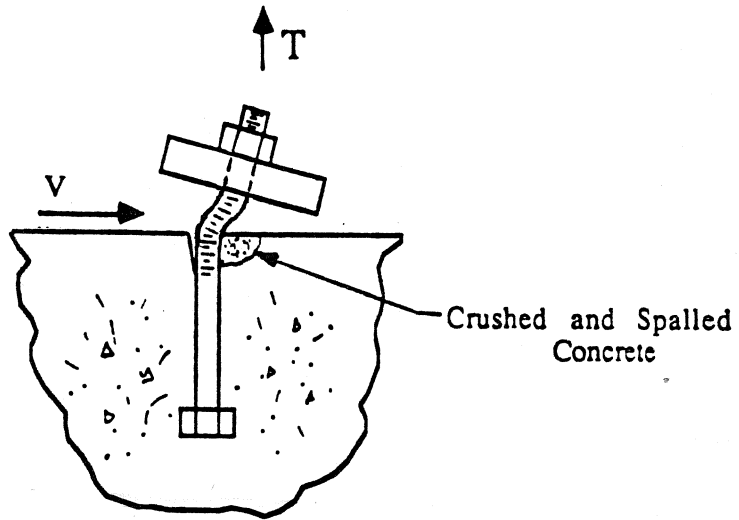


Fig. 2.3 Anchor Yielded by Combined Tension and Shear

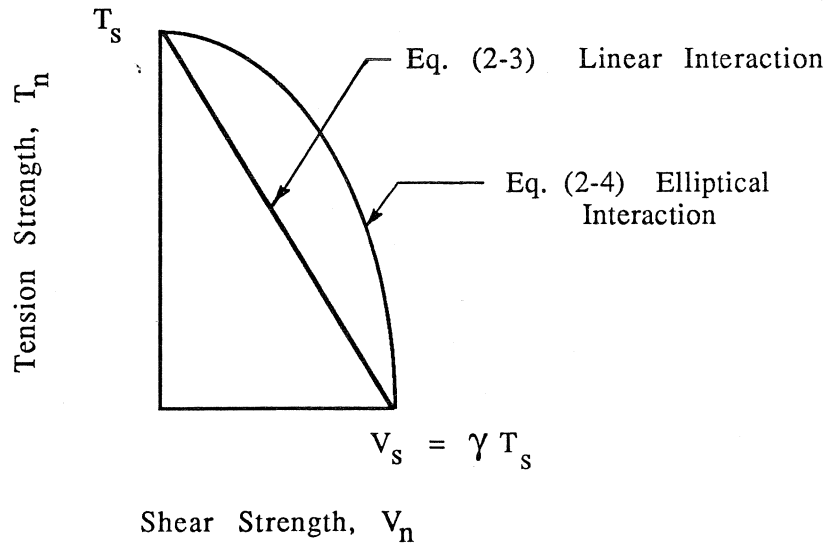


Fig. 2.4 Interaction Equations for Combined Tension and Shear

$$V_n = \gamma(T_s - T_n) \quad (2-3)$$

An elliptical interaction equation based on T_s and V_s as defined by Eq. (2-1) and Eq. (2-2) provides a more exact method for calculating the nominal strength of the steel for single anchors in tension T_n and shear V_n :

$$V_n = \gamma\sqrt{T_s^2 - T_n^2} \quad (2-4)$$

Figure 2.4 shows the curves of Eq. (2-3) and Eq. (2-4) in graphical form.

2.4 Nominal Strength of Embedment for Single Anchors

2.4.1 Nominal Tensile Strength of Embedment for Single Cast-in-Place Anchors: Two embedment failure mechanisms are possible for single cast-in-place anchors in tension:

- 1) Pullout-Cone Failure - For single anchors with an edge distance greater than or equal to $0.4 l_e$, this embedment failure mechanism is characterized by pullout of a cone of concrete radiating from the bearing surface at the head of the anchor to the surface of the concrete. The pullout-cone failure occurs when the applied tension load exceeds the tensile strength of the concrete cone. Figure 2.5 shows a typical pullout-cone failure for single cast-in-place anchors in tension.
- 2) Blowout-Cone Failure - For single anchors with an edge distance less than or equal to $0.4 l_e$, this embedment failure mechanism is characterized by blowout of a cone of concrete radiating from the embedded head of the anchor to the free edge of the concrete. The blowout-cone failure occurs when the radial bursting forces developed at the embedded anchor head exceeds the lateral resistance of the concrete, bursting or splitting the concrete adjacent to the embedded head of the anchor. Figure 2.6 shows a typical blowout-cone failure for single cast-in-place anchors in tension.

The break between pullout-cone failure and blow-out cone failure which occurs at an edge distance of $0.4 l_e$, is determined by equating the pullout-cone strength to the blowout-cone strength as defined below. The resulting equation is then solved for the edge distance m which provides a balance between these two strengths.

Pullout-Cone Failure ($m > 0.4 l_e$)

For design purposes the apex of the pullout cone shall be taken as the intersection of the anchor centerline with the far side of the anchor head. Figure 2.7 shows the design pullout cone for single cast-in-place anchors in tension. The nominal tensile strength of the embedment is determined by applying a nominal concrete tensile strength to the projected area A_p of the pullout cone at the surface of the concrete. The area of the anchor head need not be subtracted from the projected area of the cone unless its area exceeds the anchor head requirements of Section 2.7. The projected area of the pullout cone shall be limited by intersection of the cone with any free edge of concrete. Figures 2.7 and 2.8 show projected areas of the design pullout cone for single anchors.

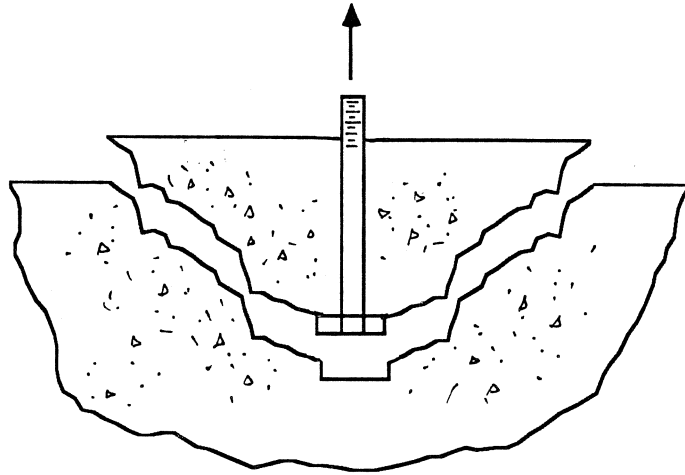


Fig. 2.5 Pullout-Cone Failure

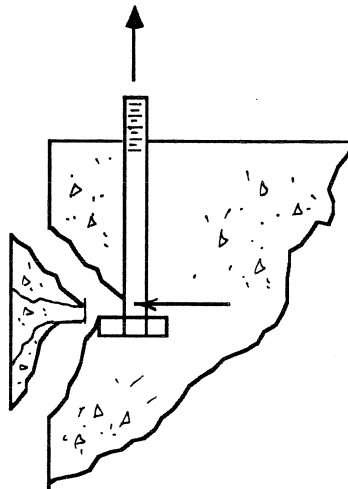


Fig. 2.6 Blowout-Cone Failure

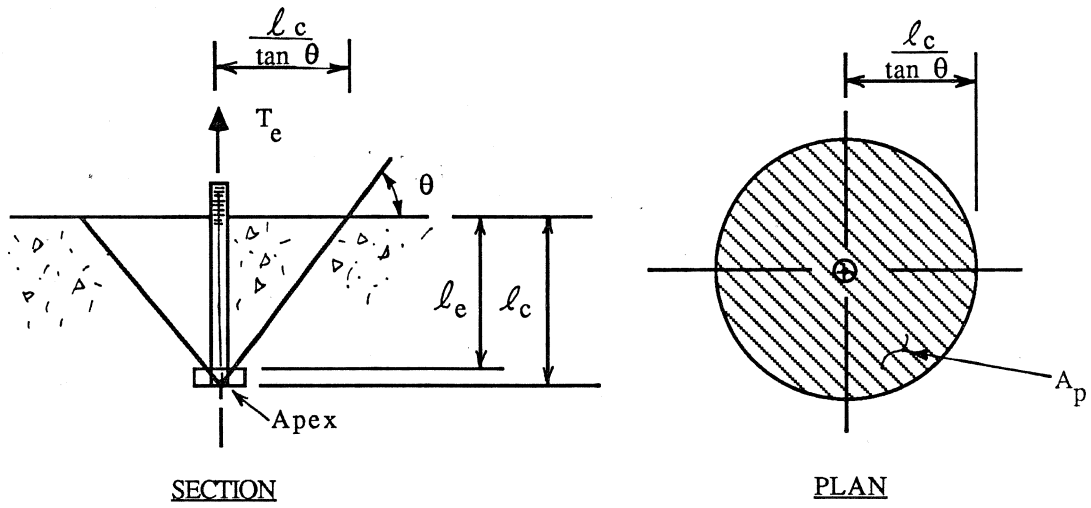


Fig. 2.7 Design Pullout Cone for Cast-in-Place Anchors

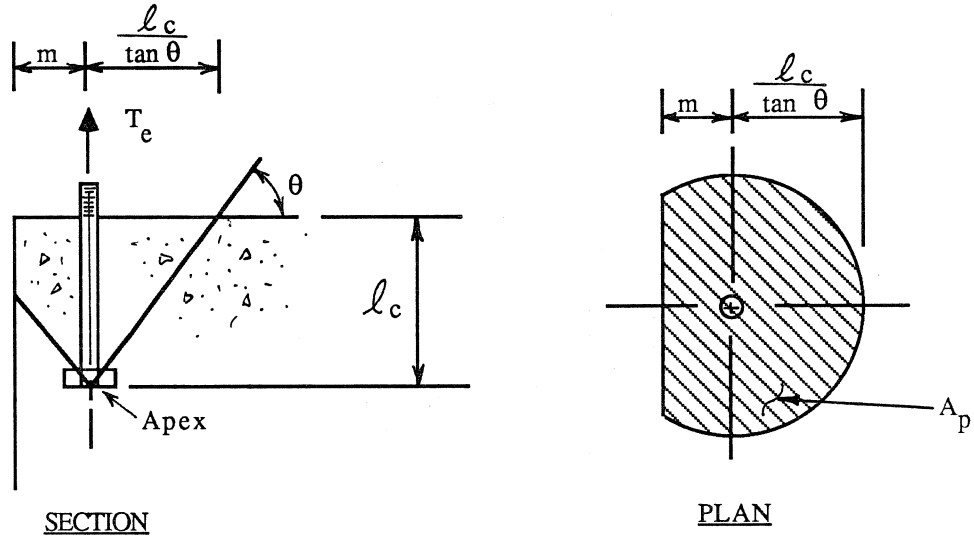


Fig. 2.8 Design Pullout Cone Limited by Free Edge

The base angle Θ of the pullout cone shall be taken as:

$$\Theta = 45^\circ \text{ for } l_e \geq 5 \text{ inches}$$

$$\Theta = 28^\circ + (3.4l_e)^\circ \text{ for } l_e < 5 \text{ inches}$$

The nominal tensile strength of the embedment for single cast-in-place anchors as governed by pullout-cone failure is given by:

$$T_e = 4\sqrt{f'_c}A_p \quad (2-5)$$

Blowout-Cone Failure ($m < 0.4 l_e$)

For design purposes the apex of the blowout cone shall be taken as the intersection of the anchor centerline and the bearing surface of the anchor head. The base angle Θ of the blowout cone shall be taken as 45° . Figure 2.9 shows the design blowout cone for single cast-in-place anchors in tension. The ratio of the lateral blowout force to the applied tensile force in the anchor is assumed to be the same as the ratio of the lateral strain in the concrete to the longitudinal strain in the concrete. The ratio of lateral force to longitudinal force shall be taken as 0.25. The nominal tensile strength of the embedment is determined by applying a nominal concrete tensile strength to the projected area A_p of the blowout cone on the free-edge surface of the concrete.

The nominal tensile strength of the embedment for single cast-in-place anchors as governed by blowout-cone failure is given by:

$$0.25T_e = 4\sqrt{f'_c}A_p$$

$$T_e = 16\sqrt{f'_c}A_p \quad (2-6)$$

2.4.2 Nominal Tensile Strength of Embedment for Single Undercut Anchors: The embedment failure mechanisms for single undercut anchors in tension are the same as for single cast-in-place anchors. For design purposes the apex of the pullout cone for undercut anchors shall be taken as the intersection of the anchor centerline with the far-side of the expansion device. Figure 2.10 shows the design pullout cone for single undercut anchors in tension. The provisions of Subsection 2.4.1 shall be used to determine the nominal tensile strength of the embedment for single undercut anchors.

2.4.3 Nominal Tensile Strength of Embedment for Single Adhesive Anchors: Two embedment failure mechanisms are possible for single adhesive anchors in tension:

- 1) Plug/Cone-Pullout Failure - For fully bonded anchors the embedment failure mechanism is characterized by pullout of an adhesive plug with a shallow concrete cone. Figure 2.11 shows a typical plug/cone-pullout failure for single adhesive anchors in tension.
- 2) Plug-Pullout Failure - For partially bonded anchors the embedment failure mechanism is characterized by pullout of an adhesive plug. This type of failure can also occur when the adhesive is improperly mixed or cured. If proper adhesive preparation is ensured by field

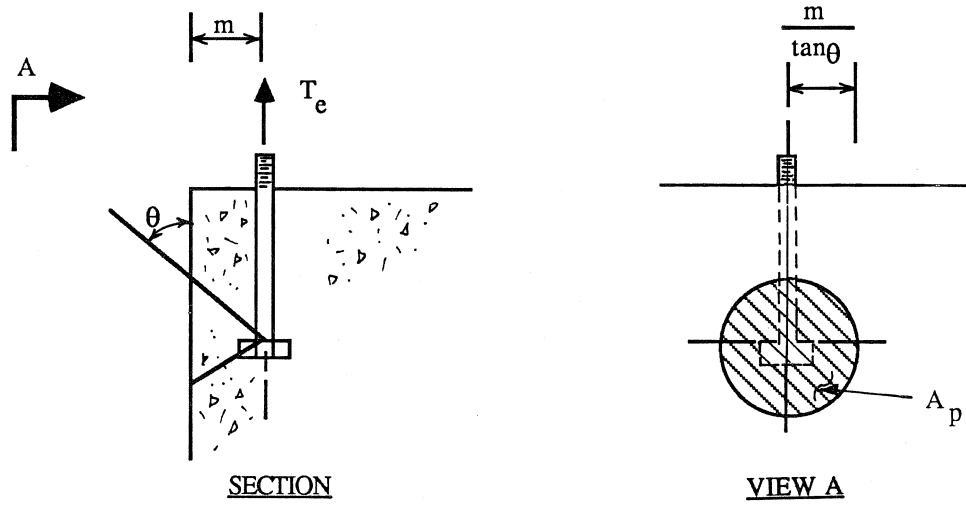


Fig. 2.9 Design Blowout Cone

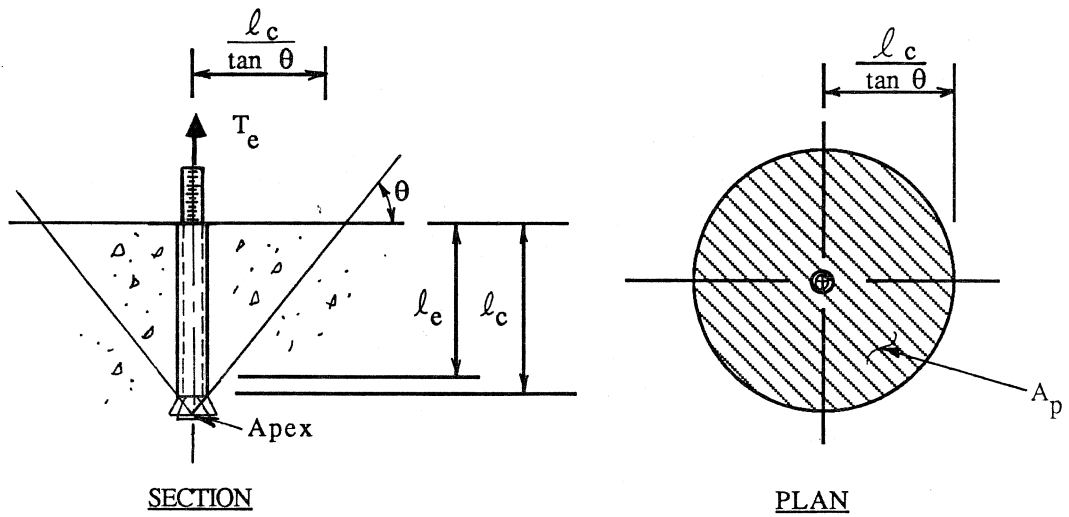


Fig. 2.10 Design Pullout Cone for Undercut Anchors

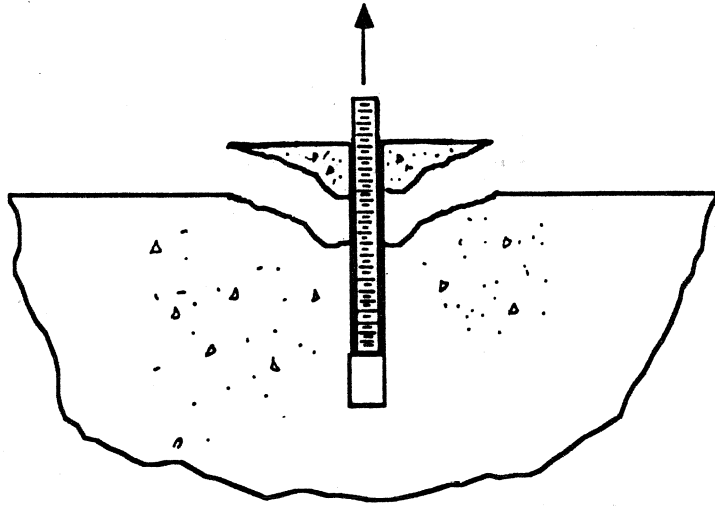


Fig. 2.11 Plug/Cone-Pullout Failure

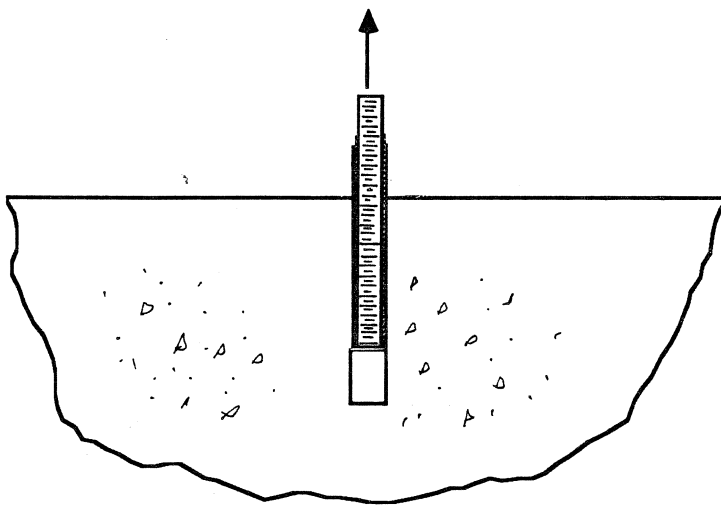


Fig. 2.12 Plug-Pullout Failure

testing and inspection, a plug-pullout failure should not occur for a fully bonded anchor. Figure 2.12 shows a typical plug-pullout failure for single adhesive anchors in tension.

There is virtually no difference between the strength of a fully bonded anchor failing by plug/cone-pullout, and that of a partially bonded anchor of the same embedment depth failing by plug-pullout.

The nominal tensile strength of the embedment for single adhesive anchors is given by:

$$T_e = \frac{\pi d_h^{1.5} u_0}{\lambda'} \tanh \left[\frac{\lambda' (l_e - 2)}{\sqrt{d_h}} \right] \quad (2-7)$$

NOTE: If $T_e \geq 0.95 \pi d_h^{1.5} u_0 / \lambda'$ the strength reduction factor ϕ_e may be taken as 0.80 for factored load design and 0.40 for unfactored load design.

2.4.4 Nominal Tensile Strength of Embedment for Single Grouted Anchors: Four embedment failure mechanisms are possible for single grouted anchors in tension:

- 1) Pullout of a concrete cone. This occurs when the edge distance is greater than or equal to $0.4 l_e$, as with a pullout-cone failure of a single cast-in-place anchor in tension.
- 2) Blowout of a concrete cone. This occurs when the edge distance is less than $0.4 l_e$, as with a blowout-cone failure of a single cast-in-place anchor in tension.
- 3) Pullout of a grout plug and shallow cone of concrete, as with a plug/cone-pullout failure of a single adhesive anchor in tension.
- 4) Pullout of the grout plug, as with a plug-pullout failure of a single adhesive anchor in tension.

The nominal tensile strength of the embedment for single grouted anchors shall be taken as the lesser of the embedment strengths for all the possible failure modes as determined by the provisions of Subsection 2.4.1 and Subsection 2.4.3.

2.4.5 Nominal Tensile Strength of Embedment for Single Expansion Anchors: Three embedment failure mechanisms are possible for single expansion anchors in tension:

- 1) Pullout of a concrete cone. This occurs when the edge distance is greater than or equal to $0.4 l_e$, as with a pullout-cone failure of a single undercut anchor in tension.
- 2) Blowout of a concrete cone. This occurs when the edge distance is less than $0.4 l_e$, as with a blowout-cone failure of a single cast-in-place anchor in tension.
- 3) Excessive slip of the anchor followed by pullout of a shallow cone of concrete. Figure 2.13 shows a typical slip/cone failure for an expansion anchor. Excessive slip of the anchor is defined as a slip greater than 5% of the embedded length after installation. A slip greater than 5% reduces the pullout-cone strength as defined in Subsection 2.4.1 by more than 10%

All expansion anchors must be tested to determine the governing embedment failure mechanism.

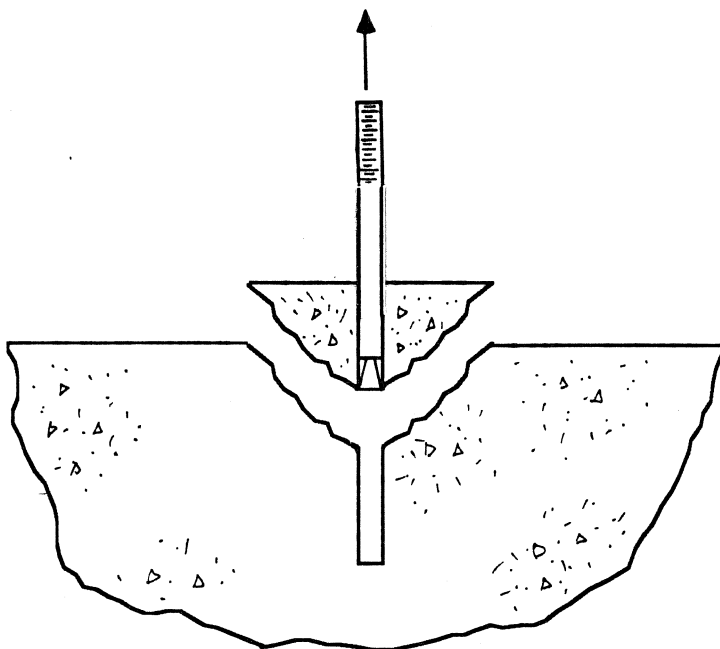


Fig. 2.13 Excessive Slip Failure with Shallow Pullout Cone

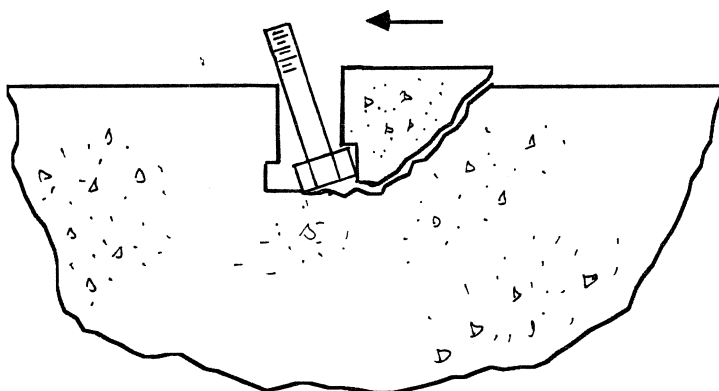


Fig. 2.14 Pullout Cone Failure

For expansion anchors which can be shown to fail by pullout-cone failure, the nominal tensile strength of the embedment shall be calculated by the methods of Subsection 2.4.1 using the same design pullout cone as for an undercut anchor.

For expansion anchors which fail by excessive slip followed by pullout of a shallow cone, the nominal tensile strength of the embedment shall be determined by testing.

2.4.6 Nominal Shear Strength of Embedment for Single Anchors: Two embedment failure mechanisms are possible for single anchors in shear:

- 1) Pryout-Cone Failure - For single anchors with an edge distance greater than or equal to $1.4 l_e$, this embedment failure mechanism is characterized by prying loose a cone of concrete on the side of the anchor away from the load. Figure 2.14 shows a typical pryout-cone failure for single anchors in shear.
- 2) Pushout-Cone Failure - For single anchors with an edge distance less than or equal to $1.4 l_e$, this embedment failure mechanism is characterized by pushout of a cone of concrete radiating from the centerline of the anchor at the surface of the concrete to the free edge. Figure 2.15 shows a typical pushout-cone failure for single anchors in shear.

The break between pryout-cone failure and push-out cone failure which occurs at an edge distance of $1.4 l_e$, is determined by equating the pryout-cone strength to the pushout-cone strength as defined below. The resulting equation is then solved for the edge distance m which provides a balance between these two strengths.

Pryout-Cone Failure ($m > 1.4 l_e$)

For anchors away from a free edge, the nominal shear strength of the embedment shall be taken as the nominal tensile strength of the embedment as determined by Eq. (2-5) or Eq. (2-7) as applicable.

The nominal shear strength of the embedment for single anchors as governed by pryout-cone failure is given by:

$$V_e = T_e \quad (2 - 8)$$

Pushout-Cone Failure ($m < 1.4 l_e$)

For design purposes the apex of the pushout cone shall be taken as the intersection of the anchor centerline with the surface of the concrete. The base angle Θ of the pushout cone shall be taken as 45° . Figure 2.16 shows the design pushout cone for single anchors in shear near a free edge. The nominal shear strength of the embedment is determined by applying a nominal concrete tensile strength to the projected area A_p of the pushout cone at the free- edge surface of the concrete. The projected area of the pushout cone shall be limited by the intersection of the cone with any other free edges of the concrete.

The nominal shear strength of the embedment for single anchors as governed by pushout-cone failure is given by:

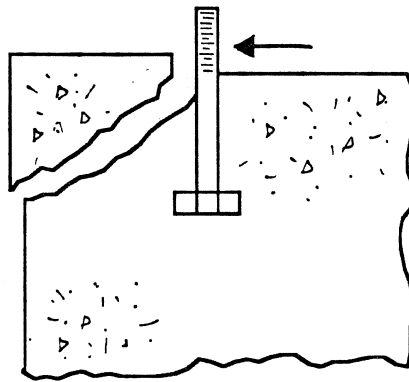


Fig. 2.15 Pushout Cone Failure

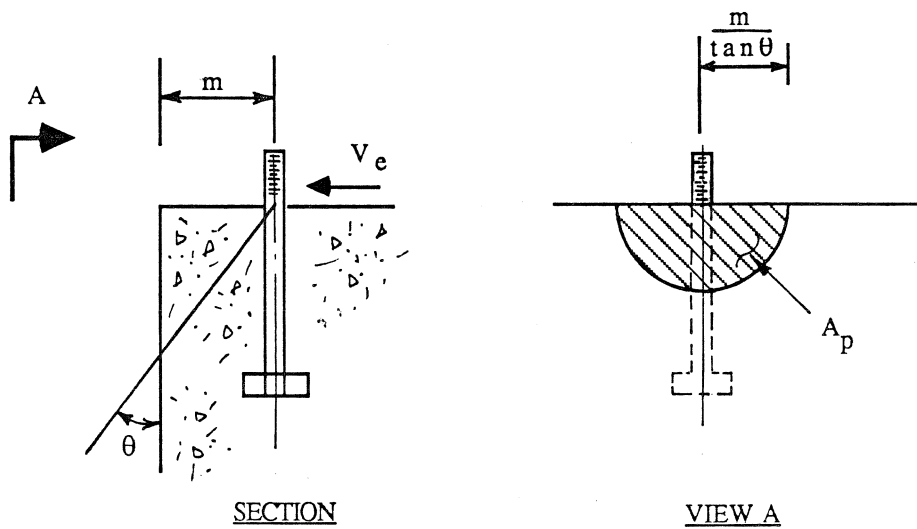


Fig. 2.16 Design Pushout Cone

$$V_e = 4\sqrt{f'_c}A_p \quad (2-9)$$

2.4.7 Nominal Strength of Embedment for Single Anchors in Tension and Shear: The nominal strength of the embedment for single anchors in tension T_n and shear V_n may be determined using a linear interaction based on T_e as defined in Subsections 2.4.1 through 2.4.5 as applicable, and V_e as defined in Subsection 2.4.6:

$$V_n = V_e\left(1 - \frac{T_n}{T_e}\right) \quad (2-10)$$

2.5 Ductile and Non-Ductile Anchors

A ductile anchor is an anchor that can be sufficiently embedded so that failure will occur by yielding and fracture of the steel when loaded in direct tension. In terms of anchor strength, a ductile anchor is an anchor which can be embedded sufficiently so that the nominal strength of the embedment is greater than or equal to the nominal strength of the steel when loaded in direct tension:

$$T_e > T_s \quad (2-11)$$

NOTES:

- 1) The nominal tensile strength of the steel T_s shall be determined by Eq. (2-1) except that F_{ut} shall be used in place of F_u .
- 2) The nominal tensile strength of the embedment T_e shall be calculated by Eq. (2-5) or Eq. (2-7) as applicable.

If an anchor cannot be sufficiently embedded to satisfy the above requirement, it is a non-ductile anchor.

Cast-in-place anchors, undercut anchors, adhesive anchors, grouted anchors, and some expansion anchors can be ductile anchors if sufficiently embedded. The use of ductile anchors in a connection does not ensure a ductile connection, but does allow for the possibility of a ductile connection.

2.6 Special Case: Quasi-Ductile Anchors

A quasi-ductile anchor is an anchor that is not embedded sufficiently to fail by yielding and fracture of the anchor, but which does fail by yielding and fracture of embedment reinforcing steel after an initial brittle failure of the concrete. Figure 2.17 shows various quasi-ductile anchors and the requirements for the embedment reinforcing steel.

The use of a quasi-ductile anchors is not recommended. If they are used, the connection design shall be in accordance with Section 5 using the nominal strength of the embedment reinforcement for T_e and/or V_e .

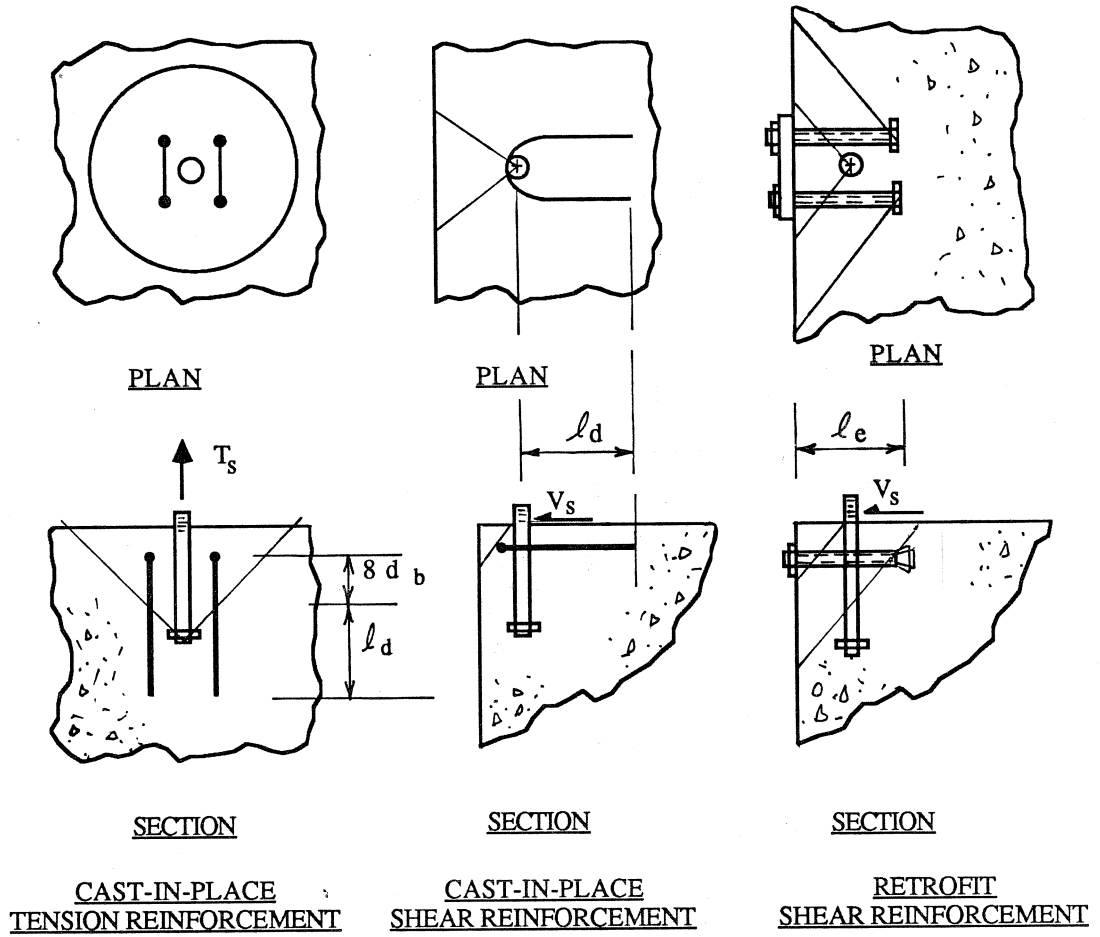


FIG. 2.17 Quasi-Ductile Anchors

2.7 Anchor Head Requirements

For all headed anchors the head shall have dimensions equivalent to a standard bolt head or standard nut. Standard dimensions for bolt heads are given in ANSI B18.2.1. Standard dimensions for nuts are given in ANSI B18.2.2.

Bearing at the anchor head does not require evaluation.

If washers or special anchor heads are used, their area shall be subtracted from the projected area of the pullout cone as defined in Section 2.4.1.

3. STEEL DESIGN FOR DUCTILE CONNECTIONS

3.1 General Requirements

Connections designed by the provisions of Section 3 shall:

- 1) Use ductile anchors as defined in Section 2.5
- 2) Use anchors of the same diameter and same material properties
- 3) Satisfy the embedment requirements of Section 4.1
- 4) Satisfy the baseplate requirements of Section 6

3.2 Design Philosophy

The design methods for ductile connections presented in Section 3 are based on the following:

- 1) Equilibrium Conditions and Compatibility of Strain - The conditions of equilibrium at the steel/concrete interface must be satisfied. Compatibility of strain at the steel/concrete interface may be neglected if the assumed location of the compressive reaction from applied moment and axial load is determined in a conservative manner.
- 2) Distribution of Tension and Shear - Tension and shear forces are redistributed among the anchors before failure. In calculating the nominal strength of a connection, every anchor in a ductile connection may be assumed to be at its maximum tensile strength as defined in Subsection 2.3.1, at its maximum shear strength as defined in Subsection 2.3.2, or at its maximum strength in combined tension and shear as defined in Subsection 2.3.3.
- 3) Shear Transfer - Shear forces are transferred by friction between the baseplate and concrete, and by bearing of the baseplate against the anchors.

Equilibrium Conditions and Compatibility of Strain

The conditions of equilibrium at the steel/concrete interface must be satisfied in all situations, but the compatibility of strain may be neglected if the compressive reaction from applied moment and axial load is located in a conservative manner.

Tests have shown that for connections satisfying the requirements of Section 3.1, the location of the compressive reaction for applied moment may be conservatively assumed to be located

at the outermost fiber of the compression element of the attached member. Figure 3.1 shows the assumed location of the compressive reaction from applied moment for various steel attachments.

If the baseplate dimensions are known, and if a portion of the baseplate extends beyond the outermost fiber of the compression element of the attached member (as shown in the first two attachments of Figure 3.1), the assumed location of the compressive reaction may be assumed to be located at a distance equal to $(m_y b_f / C)$ from the outermost fiber of the compression element of the attached member. Figure 3.2 shows the assumed location of the compressive reaction for this condition. In no case shall the location of the compressive reaction be taken outside of the boundaries of the baseplate.

For applied moment with axial load the compressive reaction, if any, may be conservatively assumed to be at the same location as for applied moment.

The theoretical distribution of compressive stresses between the baseplate and concrete is difficult to determine analytically since plane sections do not remain plane at the steel/concrete interface. The theoretical stress distribution is dependent on the relative stiffnesses of the baseplate, the anchors, and the concrete. All of these stiffnesses can be either linear or non-linear and are very difficult to define.

From a design viewpoint the theoretical strain compatibility equations are quite cumbersome and their use is not justified due to the following:

- 1) The indeterminate nature of the relative stiffnesses of the baseplate, anchors, and concrete.
- 2) The unknown variations of the actual contact surface at the steel/concrete interface due to concrete finish and warp of the baseplate.

Distribution of Tension and Shear

Tests have shown that connections satisfying the requirements specified in Section 3.1 will fully redistribute both tension and shear forces prior to failure. Every anchor in a ductile connection may be assumed to be at its maximum tensile strength as defined in Subsection 2.3.1, at its maximum shear strength as defined in Subsection 2.3.2, or at its maximum strength in combined tension and shear as defined in Subsection 2.3.3.

The actual distribution of tension and shear forces prior to full redistribution does not affect the ultimate strength of a ductile connection. Anchor forces in ductile connections redistribute as the applied load approaches the ultimate strength of the connection. As long as enough ductility is provided, each anchor obtains its maximum strength prior to failure of the connection.

Shear Transfer

Tests have shown that shear forces in steel-to-concrete connections are transferred by friction between the compression toe of the baseplate and the concrete, and by bearing of the baseplate against the anchors.

A frictional shear strength develops from the net compressive force across the steel/concrete interface due to the compressive reaction from applied moment and from direct compression. If the

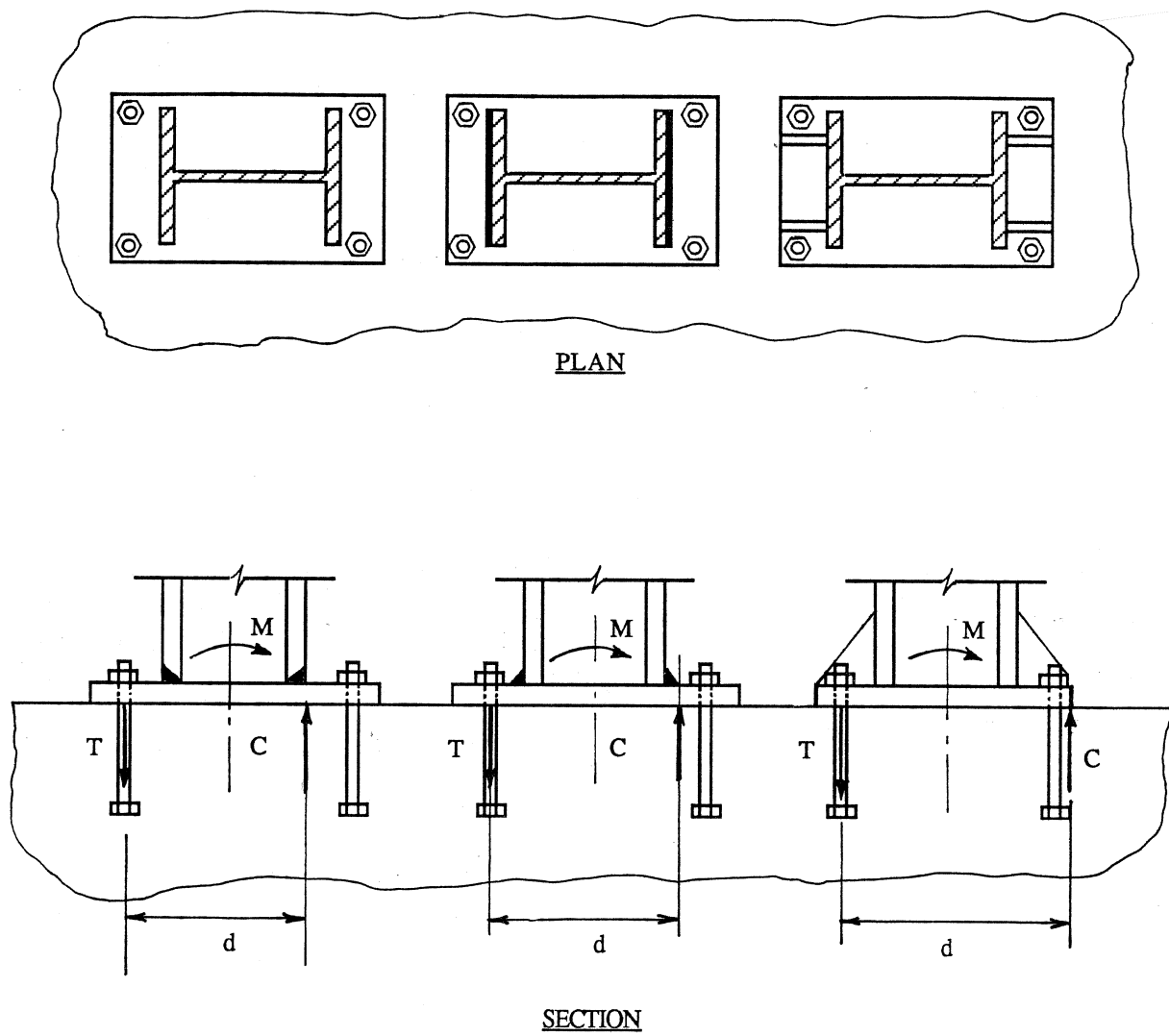


Fig. 3.1 Assumed Location of Compressive Reaction for Moment

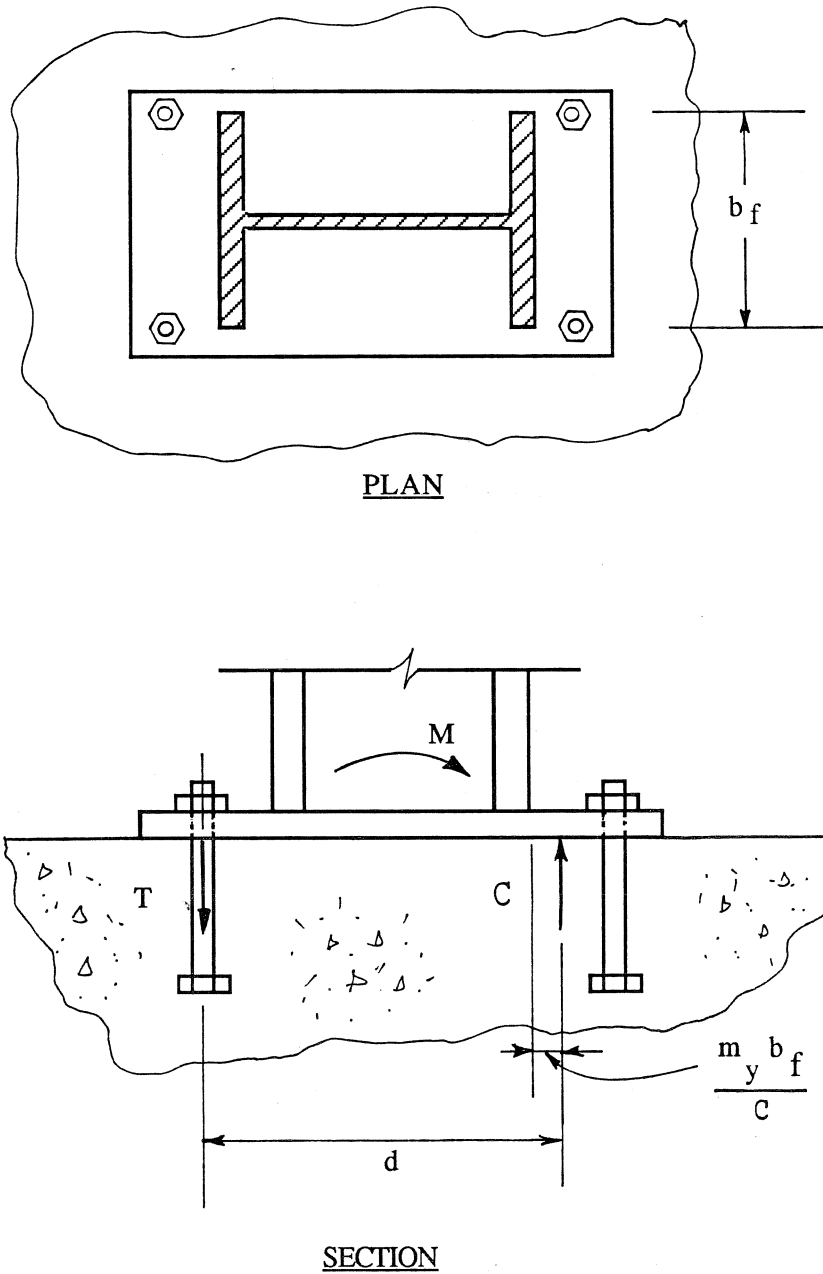


Fig. 3.2 Alternate Location of Compressive Reaction for Moment

frictional shear strength is greater than the applied shear, no shear-anchors are required. If the frictional shear strength is smaller than the applied shear the connection will slip until the baseplate begins to bear on the anchors. A sufficient amount of steel must be provided in the anchors to overcome any deficit between the applied shear and the frictional shear strength. This steel can be provided by anchors which are assumed to carry only shear, or by anchors which are assumed to carry both tension and shear.

Shear keys are much stiffer than shear-anchors and fail in a non-ductile manner. Shear keys are not recommended. Shear keys may be used in ductile connections to improve connection performance at working load levels if the total of the frictional shear strength and the shear strength provided by anchors meets the requirements of Section 3.6. If shear keys are used to improve connection performance at working load levels they must be designed to transfer the entire working load shear. Shear keys are only effective if located in the compression zone of the steel/concrete interface. The design of shear keys is not covered in this *Guide*.

3.3 Design Procedure for Moment

GENERAL PROCEDURE

- 1) Resolve all forces to the intersection of the plastic centroid of the attached member with the steel/concrete interface.
- 2) Locate the compressive reaction from the applied moment in a conservative manner as described in Section 3.2.
- 3) Locate and size anchors so that the design moment capacity of the connection is at least as large as the applied moment:

$$\phi M_n \geq M \quad (3 - 1)$$

Figure 3.3 shows the forces acting at the steel/concrete interface for a typical steel-to-concrete connection.

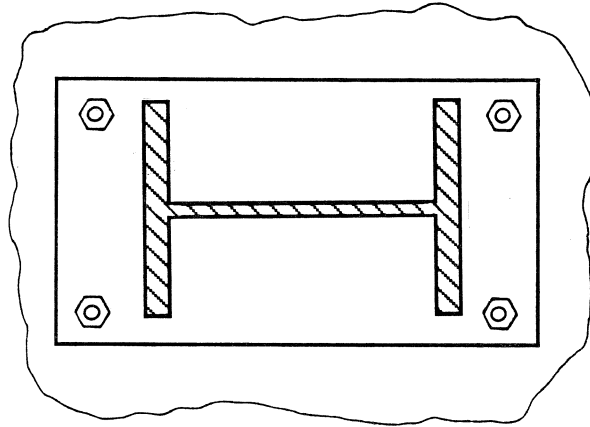
The design moment capacity is given by:

$$\phi M_n = \phi T_s d$$

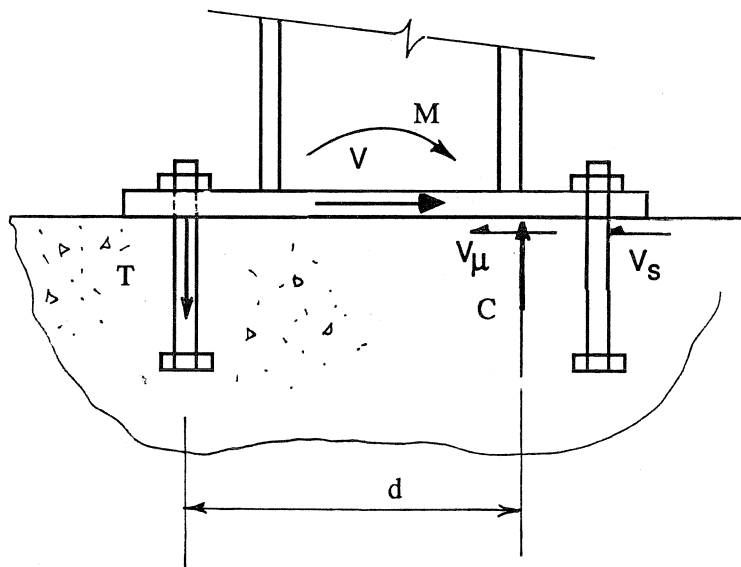
$$\phi M_n = \phi_t A_t F_u d \quad (3 - 2)$$

By equating the design moment capacity ϕM_n to the applied moment M , the minimum required area of tension steel A_t is calculated as:

$$\phi M_n = M$$



PLAN



SECTION

Fig. 3.3 Forces Acting at the Steel/Concrete Interface

$$\phi_t A_t F_u d = M$$

$$A_t = \frac{M}{\phi_t F_u d} \quad (3 - 3)$$

3.4 Design Procedure for Moment with Axial Compression

GENERAL PROCEDURE

- 1) Resolve all forces to the intersection of the plastic centroid of the attached member with the steel/concrete interface.
- 2) Locate the compressive reaction from the applied moment and axial compression in a conservative manner as described in Section 3.2.
- 3) Locate and size anchors so that the design moment capacity of the connection is at least as large as the applied moment less any moment capacity provided by the applied axial compression.

3.5 Design Procedure for Moment with Axial Tension

GENERAL PROCEDURE

- 1) Resolve all forces to the intersection of the plastic centroid of the attached member with the steel/concrete interface.
- 2) Locate the compressive reaction, if any, from the applied moment and axial tension in a conservative manner as described in Section 3.2.
- 3) Locate and size anchors so that the design tension and moment capacities of the connection are at least as large as the applied axial tension and moment.

3.6 Design Procedure for Shear

GENERAL PROCEDURE

- 1) Resolve all forces to the intersection of the plastic centroid of the attached member with the steel/concrete interface.
- 2) Determine the total compressive reaction C from the applied moment and axial load.
- 3) Determine if the design frictional shear strength $\phi_e V_\mu$ is greater than or equal to the applied shear V. If $\phi_e V_\mu$ is greater than or equal to V, no shear steel is required. If $\phi_e V_\mu$ is less than V, locate and size anchors so that the design shear strength of the connection is at least as large as the applied shear:

$$\phi V_n \geq V \quad (3 - 4)$$

Figure 3.2 shows the shear forces acting at the steel/concrete interface for a typical steel-to-concrete connection.

The design shear strength is given by:

$$\phi V_n = \phi(V_\mu + V_s)$$

$$\phi V_n = \phi_e \mu C + \phi_v \gamma A_v F_u \quad (3-5)$$

By equating the design shear strength ϕV_n to the applied shear V , the minimum required area of shear steel A_v is calculated as:

$$\phi V_n = V$$

$$\phi_e \mu C + \phi_v \gamma A_v F_u = V$$

$$A_v = \frac{V - \phi_e \mu C}{\phi_v \gamma F_u} \quad (3-6)$$

Note: When $V \leq \phi_e \mu C$ no shear steel is required.

For the case of applied moment and shear with no axial load $C = (M/d)$, and the minimum area of shear steel may be calculated as:

$$A_v = \frac{V - \phi_e \mu M/d}{\phi_v \gamma F_u} \quad (3-7)$$

Note: When $V \leq \phi_e \mu M/d$ no shear steel is required.

4. EMBEDMENT REQUIREMENTS FOR DUCTILE CONNECTIONS

4.1 General Requirements

A ductile connection is a connection that is sufficiently embedded so that failure occurs by yielding and fracture of the steel. For the purposes of this *Guide*, a ductile connection is a connection in which the design strength of the embedment exceeds the nominal strength of the steel:

$$\phi_e T_{ec} > T_{sc} \quad (4-1)$$

and

$$\phi_e V_{ec} > V_{sc} \quad (4-2)$$

Eq. (4-1) and Eq. (4-2) must be satisfied for a ductile connection. Section 4.3 provides methods for calculating the nominal strength of the steel for ductile connection embedment design, T_{sc} and V_{sc} . Section 4.4 provides methods for calculating the nominal strength of the embedment for a connection, T_{ec} and V_{ec} .

4.2 Design Philosophy

The embedment requirements for ductile connections as expressed by Eq. (4-1) and Eq. (4-2) guarantee steel failure prior to embedment failure. As discussed in Section 1.2, steel failure is preferred since load redistribution and energy absorption occur prior to failure.

4.3 Nominal Strength of Steel for Ductile Connection Embedment Design

4.3.1 Nominal Tensile Strength of Steel for Ductile Connection Embedment Design: For embedment design the nominal tensile strength of the steel shall be taken as the total nominal tensile strength of all the tension-anchors in the connection. A tension-anchor is any anchor that is assumed to be in tension in the steel design of Section 3, regardless of the actual amount of tension the anchor is assumed to carry. It is not sufficient to use the required area of tension steel as determined by applied loads for the embedment design. The nominal tensile strength of the steel for ductile connection embedment design shall be taken as:

$$T_{sc} = A_{tc} F_{ut} \quad (4-3)$$

4.3.2 Nominal Shear Strength of Steel for Ductile Connection Embedment Design: For embedment design the nominal shear strength of the steel shall be taken as the total nominal shear strength of all the shear-anchors in the connection. A shear-anchor is any anchor that is assumed to be in shear in the steel design of Section 3 regardless of the actual amount of shear the anchor is assumed to carry. It is not sufficient to use the required area of shear steel as determined by applied loads for the embedment design. The nominal shear strength of the steel for ductile connection embedment design shall be taken as:

$$V_{sc} = \gamma A_{vc} F_{ut} \quad (4-4)$$

4.4 Nominal Strength of Embedment for Connections

4.4.1 Nominal Tensile Strength of Embedment for Connections with Cast-in-Place Tension-Anchors: The nominal tensile strength of the embedment for a connection with cast-in-place tension-anchors is determined in the same manner as the nominal tensile strength of the embedment for a single cast-in-place anchor in tension as given in Subsection 2.4.1. A nominal concrete tensile strength is applied to the projected area of assumed failure cones radiating from the embedded heads of the tension-anchors. The assumed failure cones can be from either:

- 1) Pullout-Cone Failure
- 2) Blowout-Cone Failure

The nominal tensile strength of the embedment for connections with cast-in-place tension-anchors is given by the lesser of Eq. (4-5) and Eq. (4-6).

Pullout-Cone Failure

For pullout failure the projected area of the failure surface shall be limited by the overlap of individual tension-anchor pullout cones, by the intersection of the individual tension-anchor pullout cones with any free edge of concrete, and by the overall thickness of the concrete. The geometry of the individual pullout cones shall be as defined in Subsection 2.4.1. Figure 4.1 shows projected areas for overlapping cones. Figure 4.2 shows the projected area as limited by the concrete thickness.

An exact calculation of the projected area of the pullout failure surface for overlapping failure cones is difficult, tedious, and not justified given the inexact nature of other parameters in the embedment design. Section 4.5 provides approximate methods for evaluating the projected area of the pullout failure surface for embedment design.

The nominal tensile strength of the embedment for connections with cast-in-place tension-anchors as governed by pullout- cone failure is given by:

$$T_{ec} = 4\sqrt{f'_c}A_{pg} \quad (4-5)$$

Blowout-Cone Failure

For blowout failure the projected area of the failure surface shall be limited by the overlap of individual tension-anchor blowout cones and by the intersection of the individual tension-anchor blowout cones with any free edge of concrete. The geometry of the individual blowout cones shall be as defined in Subsection 2.4.1.

The projected area of the blowout failure surface for overlapping failure cones may be determined by the approximate methods given in Section 4.5.

The nominal tensile strength of the embedment for a connection with cast-in-place tension-anchors as governed by blowout- cone failure is given by:

$$T_{ec} = 16\sqrt{f'_c}A_{pg} \quad (4-6)$$

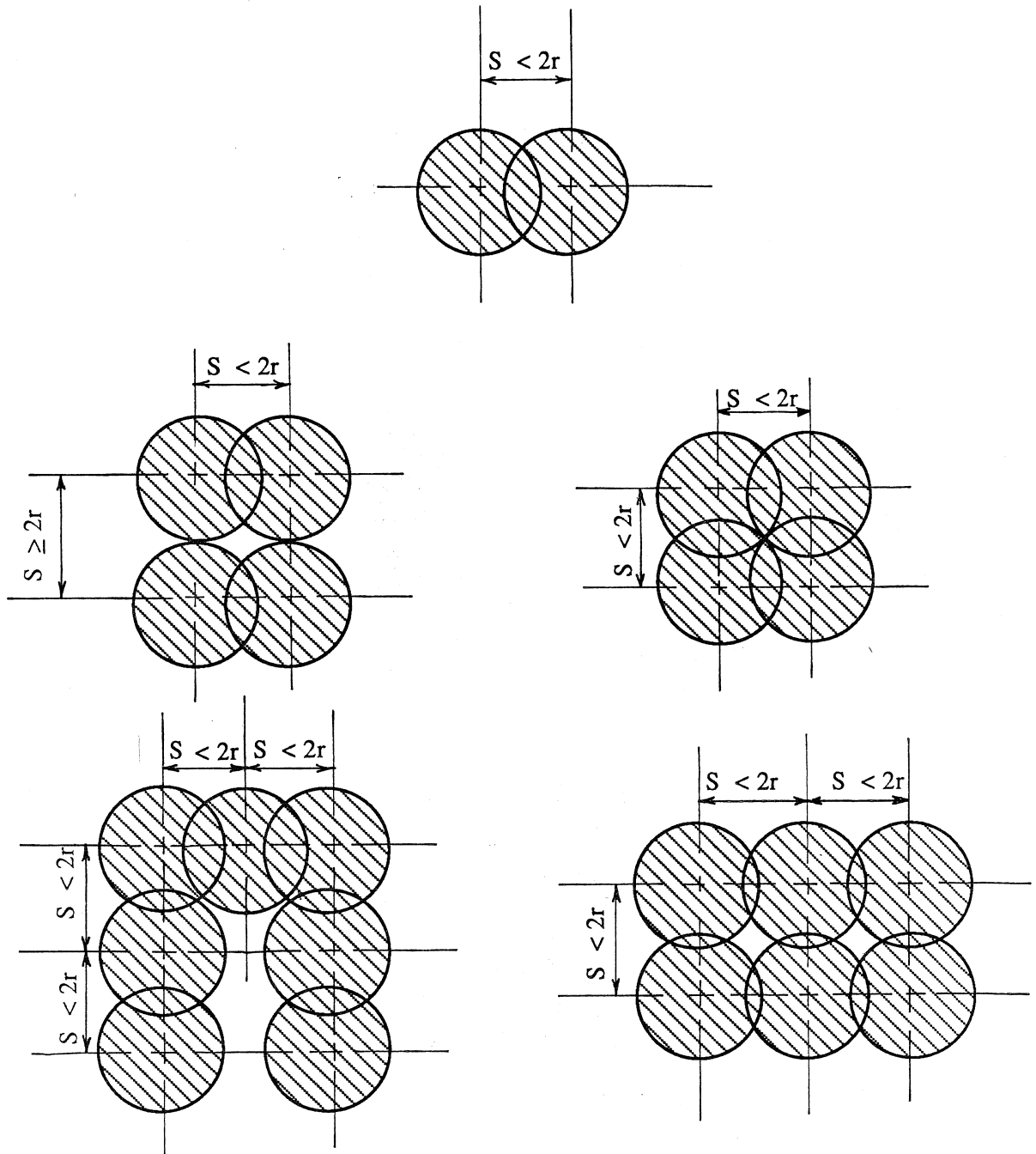


Fig. 4.1 Projected Areas for Overlapping Cones

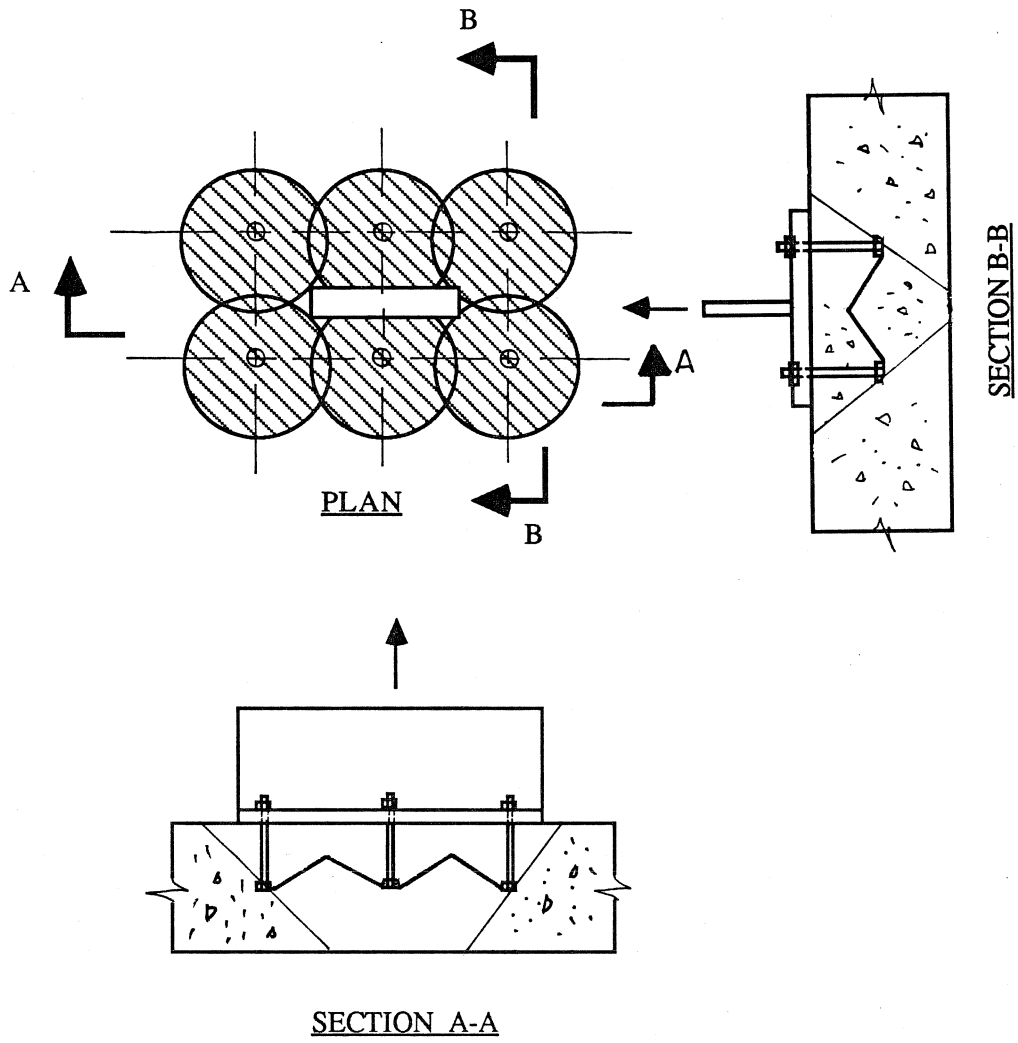


Fig. 4.2 Projected Area as Limited by Concrete Thickness

4.4.2 Nominal Tensile Strength of Embedment for Connections with Undercut Tension-Anchors: The nominal tensile strength of the embedment for a connection with undercut tension-anchors shall be determined in the same manner as for a connection with cast-in-place tension-anchors as given in Subsection 4.4.1 except that the effective embedded length shall be as specified in Subsection 2.4.2.

4.4.3 Nominal Tensile Strength of Embedment for Connections with Adhesive Tension-Anchors: The nominal tensile strength of the embedment for a connection with adhesive tension-anchors is determined in a similar manner as the nominal tensile strength of a single adhesive anchor in tension as given in Subsection 2.4.3. The failure mode can be from either:

- 1) Plug/Cone-Pullout Failure - For fully bonded adhesive anchors
- 2) Plug-Pullout Failure - For partially bonded adhesive anchors

The nominal tensile strength of the embedment for connections with either fully bonded or partially bonded adhesive tension-anchors is given by:

$$T_{ec} = nT_e \quad \text{For } s \geq 8 \text{ inches} \quad (4-8)$$

NOTE: For $s < 8$ inches, $T_{ec} = 0.85nT_e$

4.4.4 Nominal Tensile Strength of Embedment for Connections with Grouted Tension-Anchors: The nominal tensile strength of the embedment for a connection with grouted tension-anchors shall be taken as the lesser of the strengths given in Subsection 4.4.1 and Subsection 4.4.3.

4.4.5 Nominal Tensile Strength of Embedment for Connections with Expansion Tension-Anchors: For a connection with expansion tension-anchors which can be shown to fail by pullout-cone failure, the nominal tensile strength of the embedment for the connection shall be calculated by the methods given in Subsection 4.4.1 using the same effective embedded length as specified for an undercut anchor in Subsection 2.4.2.

For a connection with expansion tension-anchors that are spaced further than l_e apart and which fail by excessive slip (greater than 5% of the embedded length after installation) followed by pullout of a shallow cone, the nominal tensile strength of the embedment for the connection shall be taken as the sum of the individual tension-anchor embedment strengths as determined by testing. For expansion anchors spaced closer than l_e , the nominal tensile strength of the embedment for the connection shall be determined by testing of closely spaced expansion anchors.

4.4.6 Nominal Shear Strength of Embedment for Connections with Shear-Anchors: The nominal shear strength of the embedment for a connection with shear-anchors is determined in the same manner as the nominal shear strength of a single anchor in shear as given in Subsection 2.4.6 adjusted for failure-cone overlap and edge conditions. The failure mode can be from either:

- 1) Pryout-Cone Failure
- 2) Pushout-Cone Failure

The nominal shear strength of the embedment for a connection with shear-anchors is given by the lesser of Eq. (4-9) or Eq. (4-10):

Pryout-Cone Failure

For shear-anchors away from a free edge the nominal shear strength of the embedment for the connection shall be calculated in the same manner as the nominal tensile strength of the embedment as determined by Eq. (4-5) or Eq. (4-8) as applicable.

The nominal shear strength of the embedment for a connection with shear-anchors as governed by pryout-cone failure is given by:

$$V_{ec} = T_{ec} \quad (4-9)$$

Pushout-Cone Failure

For shear-anchors near a free edge of concrete a nominal concrete tensile strength is applied to the projected area of assumed pushout failure cones radiating from the centerlines of the anchors at the surface of the concrete to the free edge. For pushout failure the projected area of the failure surface shall be limited by the overlap of individual anchor pushout cones and by the intersection of the individual anchor pushout cones with any free edge of concrete. The geometry of the individual pushout cones shall be as defined in Subsection 2.4.6.

The projected area of the pushout failure surface for overlapping failure cones may be determined by the approximate methods given in Section 4.5.

The nominal shear strength of the embedment for connections with shear-anchors as governed by pushout-cone failure is given by:

$$V_{ec} = 4\sqrt{f'_c} A_{pg} \quad (4-10)$$

4.5 Calculation of Projected Area for Overlapping Failure Cones

Exact calculation of the projected area for overlapping failure cones is difficult and cannot be justified given the inexact nature of other parameters in the embedment design, such as the concrete tensile strength and the shape of the failure cones. The approximate method given in this section is generally conservative, and in the few situations where it is unconservative the error is less than 2%.

The approximate method is based on connecting the overlapping failure cones by tangents, calculating the resulting projected area by relatively simple formulas, and then modifying that projected area by a reduction factor. Figure 4.3 shows approximate projected areas A_{pa} for overlapping cones. The reduction factor β is given by:

$$\beta = 1 - 0.05\alpha^2 \quad (4-11)$$

where α is the ratio of the largest anchor spacing between adjacent anchors in a group of anchors with overlapping failure cones, to the radius of an individual failure cone. The factor α will always

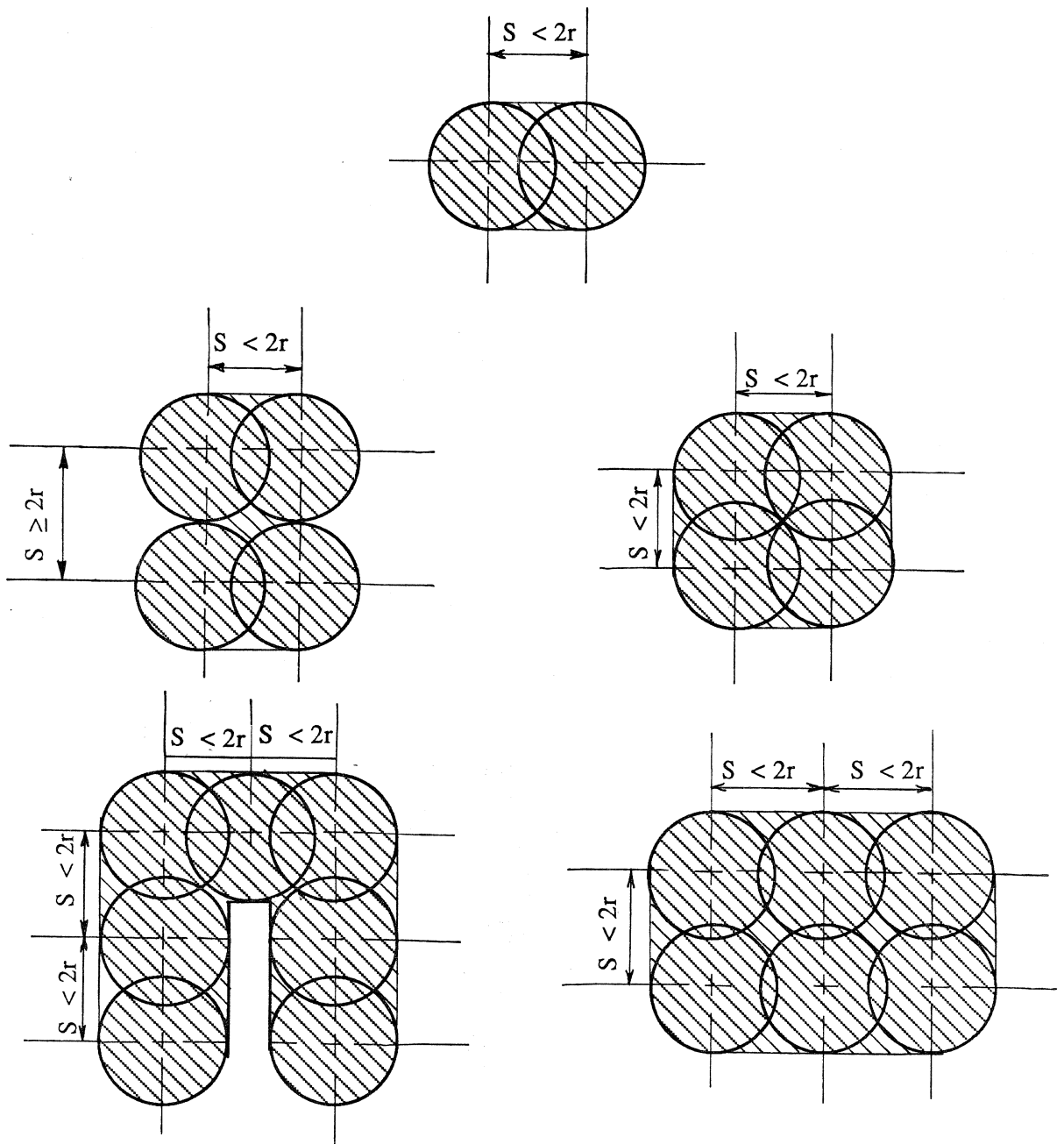


Fig. 4.3 Approximate Projected Areas for Overlapping Cones

be less than 2 for overlapping failure cones (when α is greater than or equal to 2 the failure cones do not overlap).

For design purposes the effective projected area for groups of anchors may be taken as:

$$A_{pg} = \beta A_{pa} \quad (4 - 12)$$

5. DESIGN OF NON-DUCTILE CONNECTIONS

5.1 General Requirements

In designing non-ductile connections, the following requirements must be met:

- 1) Satisfy the applicable conditions of equilibrium and compatibility of strain at the steel/concrete interface.
- 2) Assume no redistribution of tension among the anchors.
- 3) Assume that no more than two anchors carry shear. No shear anchors are required for connection shear strength if the applied shear V is less than the design frictional shear strength $\phi_e \mu C$.
- 4) Limit the maximum load on any one anchor to the single-anchor design strength as determined by the following:
 - a) For anchors without overlapping failure cones the single-anchor design strength shall be taken as T_e or V_e as defined in Section 2.4 multiplied ϕ_e .
 - b) For anchors with overlapping failure cones the single-anchor design strength T_e or V_e shall be taken as T_{ec} or V_{ec} as defined in Section 4.4 divided by the number of anchors used to determine T_{ec} or V_{ec} multiplied by ϕ_e .
- 5) Satisfy the baseplate requirements of Section 6.

5.2 Design Philosophy

The actual distribution of compressive stresses between the baseplate and concrete is impossible to determine analytically in a design situation since unknown variations in the actual steel/concrete contact surface cannot be accounted for in the design.

It is acceptable to use a transformed section design approach (linear variation of stress and strain) for non-ductile connections since experience has shown that connections designed by this procedure behave in an satisfactory manner. The transformed section design method is not "correct" since plane sections do not remain plane at the steel/concrete interface and since unknown variations in the actual steel/concrete contact surface cannot be accounted for.

Note that when anchors in non-ductile connections have overlapping failure cones their strength is dependent on their spacing. In order to determine the required spacing of the anchors for moment resistance an assumption must be made regarding the strength of individual anchors. If the assumed strength is greater than the strength determined by Section 4.4 for the selected spacing the design must be repeated using the lower strength. In short, the design of non-ductile connections can be a lengthy, repetitive process.

Shear keys may be used in non-ductile connections but are not recommended. If shear keys are used they must be designed to transfer the entire shear. Shear keys are only effective if located in the compression zone of the steel/concrete interface. The design of shear keys is not covered in this design guide.

The design requirements given in Section 5.1 are believed to provide a sufficiently conservative design for non-ductile connections.

6. DESIGN OF BASEPLATES

6.1 General Requirements

All connections designed using the provisions of Section 3 or Section 5 shall have baseplates which satisfy the requirements of Section 6.

6.2 Design Philosophy

The design methods for baseplates presented in Section 6 are based on the following requirements:

- 1) Baseplate Flexibility - The portion of the baseplate which transmits tension loads from the attached member to the tension-anchors shall not develop a plastic hinge. The portion of the baseplate which transmits compression loads from the attached member to the concrete may develop a plastic hinge.
- 2) Concrete Bearing - Bearing of the baseplate on the concrete shall not cause a failure in the concrete.
- 3) Steel Bearing - The baseplate shall be of sufficient thickness so that bearing of the anchors on the baseplate does not affect the strength of the connection.
- 4) Anchor Holes - The size of anchor holes in the baseplate shall be limited to a reasonable oversize for redistribution of shear among the anchors. The spacing and edge distance for anchor holes shall be sufficient so that the baseplate does not tear.

Baseplate Flexibility

The procedures for anchor design given in Section 3 and Section 6 are based on assuming a conservative location of the compressive reaction from applied moment and axial load. Tests have shown that the assumed location of the compressive reaction as described in Section 3.2 is conservative for rigid and “reasonably” flexible baseplates.

A “reasonably” flexible baseplate is a baseplate in which that portion of the baseplate which transmits tension loads from the attached member to the tension-anchors does not develop a plastic hinge. The portion of the baseplate which transmits compression loads from the attached member to the concrete may develop a plastic hinge. Fig. 6.1 shows the typical deformed shape of a “reasonably” flexible baseplate.

If a baseplate is more flexible than the “reasonably” flexible baseplate described above prying action may develop and increase the tensile forces in the anchors. Additionally the stiffness of the connection will be significantly reduced since hinges will form in the baseplate on both the tension side and the compression side of the connection. Connections with baseplates which do not meet the minimum strength requirements for a “reasonably” flexible baseplate as defined in Section 6.3 and Section 6.4 may behave in a satisfactory manner in some situations but they are not covered in this design guide.

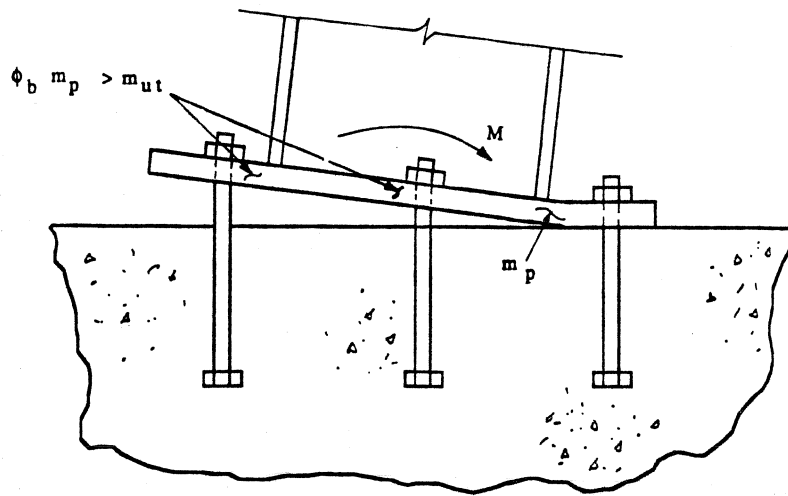


Fig. 6.1 Typical Deformed Shape of “Reasonably” Flexible Baseplate

Concrete Bearing

Tests have shown that the localized bearing stresses on confined concrete can be as high as $6 f'_c$ with no adverse affects on the ultimate strength of the connection. In the absence of more extensive tests, the bearing strength provisions used in this *Guide* are based on a traditional design approach.

Steel Bearing

Tests have shown that bearing stresses between the shear-anchors and the baseplates which cause deformation around the hole do not affect the connection strength. The bearing strength provisions used in this guide prevent excessive deformations around the anchor hole.

Anchor Holes

Tests have shown that holes $1/4$ inch larger than the anchors allow full redistribution of shear in multiple-anchor connections.

The minimum anchor hole spacing and edge distance used in this guide are the same as those normally used for steel plates.

6.3 Baseplate Design Procedure for Flexure

The baseplate shall be of a sufficient thickness so that the design moment capacity per unit width of the baseplate exceeds the maximum moment per unit width induced in the baseplate by the tension- anchors:

$$\phi_b m_p \geq m_{ut} \quad (6-1)$$

By equating the design moment capacity to the maximum moment induced in the baseplate by the tension-anchors, the minimum required plate thickness can be calculated:

$$\frac{\phi_b F_y t^2}{4} = m_{ut}$$

$$t = \sqrt{\frac{4m_{ut}}{\phi_b F_y}} \quad (6-2)$$

Any rational design procedure, including yield line analysis, may be used to evaluate m_{ut} .

6.4 Baseplate Design Procedure for Shear

The baseplate shall be of a sufficient thickness so that the design shear strength per unit width of the baseplate exceeds the maximum shear per unit width induced in the baseplate by the tension- anchors:

$$\phi_v v_n \geq v_{ut} \quad (6-3)$$

By equating the design shear strength to the maximum shear induced in the baseplate by the tension-anchors, the minimum required plate thickness can be calculated:

$$\phi_v 0.6 F_y t = v_{ut}$$

$$t = \frac{v_{ut}}{\phi_v (0.6 F_y)} \quad (6-4)$$

Any rational design procedure may be used to evaluate v_{ut} .

6.5 Baseplate Design Procedure for Concrete Bearing

The design bearing strength of the concrete shall not be less than the compressive reaction from the applied moment and axial load as determined by Section 3 or Section 5:

$$\phi_p P_n \geq C \quad (6-5)$$

The design bearing strength of the concrete is given by:

$$\phi_p P_n = \phi_p 0.85 f'_c A_1 \sqrt{A_2/A_1} \quad (6-6)$$

where:

$$\sqrt{A_2/A_1} \leq 2$$

6.6 Baseplate Design Procedure for Steel Bearing

The design bearing strength of an anchor hole in the baseplate shall be greater than or equal to the average nominal shear strength of a shear-anchor:

$$\phi_p P_n \geq \gamma A_s F_{ut} \quad (6-7)$$

The design bearing strength of an anchor hole in the baseplate is given by:

$$\phi_p P_n = \phi_p 2.4 d_b t F_u \quad (6-8)$$

By equating the design bearing strength to the average nominal shear strength of a shear-anchor the minimum required baseplate thickness for bearing is calculated as:

$$\phi_p 2.4 d_b t F_u = \gamma A_s F_{ut}$$

$$t = \frac{\gamma A_s F_{ut}}{\phi_p 2.4 d_b F_u} \quad (6-9)$$

Note that in Eq. (6-9) F_{ut} is for the anchor steel while F_u is for the baseplate steel.

6.7 Baseplate Anchor Hole Requirements

Anchor holes for shear-anchors shall be no larger than $d_b + 1/4$ inch.

The minimum edge distance from the centerline of an anchor hole to the edge of the baseplate shall be $1.5 d_b$.

The center-to-center distance between anchor holes shall not be less than $3 d_b$.

APPENDIX A INSTALLATION REQUIREMENTS FOR RETROFIT ANCHORS

In this Appendix, installation procedures for retrofit anchors are described. These retrofit anchors include adhesive anchors, grouted anchors, expansion anchors, and undercut anchors.

A.1 Installation of Adhesive Anchors

To ensure the satisfactory performance of an adhesive connection, several requirements shall be met in the installation of the anchor. The areas of interest are preparation of the anchor, preparation of the hole, preparation of the adhesive, placement of the adhesive and curing of the adhesive.

A.1.1 Anchor Preparation. Threaded rod should be used for the anchor. The threaded rod should be cut to the desired length prior to installation. Prior to installation, the anchor should also be thoroughly cleaned by wire-brushing, immersing in a solvent such as methyl-ethyl-ketone, and wiping clean.

A.1.2 Preparation of the Hole. Unless otherwise specified by the adhesive manufacturer, the hole diameter should be 1/8-inch larger than the anchor diameter. Holes should be drilled with a rotary hammer drill. When drilling the holes, care should be taken to maintain a straight hole. Hole depths should be measured after cleaning dust from the bottom of the hole. After drilling, the hole should be cleaned with a stiff fiber brush (not a wire brush) to remove as much dust as possible from the sides of the hole. The hole should then be vacuumed with an attachment long enough to reach the bottom of the hole, and having slits along its length to vacuum the sides of the hole. Fingers should not be placed in the hole, as oils from the fingers will coat the sides of the hole. To check for residual dust, place a clean, gloved finger in the hole.

While some manufacturers recommend cleaning the hole with compressed air instead of a vacuum, other research suggests that vacuuming is preferable.

If specified, bond breaker should be applied to the top two inches of the hole. The easiest method is to use two-inch wide duct tape on the anchor and on the inside of the hole. Another possible bond breaker is silicone sealant, placed on the anchor and on the inside of the hole. If glass-capsule products are being used, the top two inches of the hole should be drilled with a slightly larger drill bit, to ensure that the broken glass does not cut up the bond breaker.

A.1.3 Preparation of Adhesive. Methods for mixing adhesives vary from manufacturer to manufacturer and adhesive to adhesive. Mixing can be either automatic or by hand. Special attention should be given to each manufacturer's recommended temperature ranges for placement of their adhesive. If the adhesive is too warm, its pot life will be drastically shortened. The effects of high ambient temperatures can be abated by refrigerating the components prior to use.

Epoxies and vinyl ester adhesives are available either in a "gun" applicator, or with the resin and catalyst in separate containers. When mixing epoxies by hand, measurements should be very carefully made and the manufacturers instructions followed closely. Epoxies should be mixed with a paddle-type paint mixer ("Jiffy" or similar) attached to a rotary drill and turned at low

speed. Properly mixed adhesive will have a uniform color. Use of a “gun”-type applicator reduces the possibility of improper proportioning. These “guns” automatically mix and proportion the two components of the adhesive. When starting a new tube of adhesive, the mixed adhesive should be pumped out for discard until it is of a uniform color, indicating the proper mixture. The first and last portions of a tube can be improperly mixed due to one component entering the mixing tube before the other.

Polyester resins can be supplied either as separate two-component systems to be mixed by hand, or as ready-to-use glass capsules. The hand-mixed systems usually consist of a can of resin and a pre-measured packet of catalyst. The catalyst should be added to a full can of resin and mixed according to the manufacturer’s recommendations. The glass capsules require a angle-tipped anchor. The capsules are dropped into the prepared hole and broken with the anchor. The anchor is forced into the hole using a rotary drill. This process breaks the capsule up and mixes the resin and catalyst. The manufacturer’s directions should be carefully followed.

A.1.4 Placement of Anchors. When placing adhesive in the prepared hole, care should be taken to avoid trapped air bubbles. With a gun-type applicator, the adhesive should be placed from the bottom of the hole to the top, slowly moving the gun up until the hole is 1/3 to 1/2 full. Hand-mixed adhesives should be poured into the hole until the hole is 1/3 to 1/2 full. The anchor should then be pushed into the hole while rotating the anchor to break up any air pockets.

Special consideration is necessary for horizontal or overhead installations. Adhesives used in these installations should be of high viscosity so as not to leak out of the hole during installation.

A.1.5 Curing of Adhesives. The manufacturer’s guidelines should be followed for curing conditions.

A.2 Grouted Anchors

There are many similarities between the placement of grouted anchors and adhesive anchors. Principal differences include hole preparation and grout preparation.

A.2.1 Anchor Preparation. Anchors should be prepared in the same manner as for adhesive anchors. The anchor should be wire- brushed and cleaned off with a solvent.

A.2.2 Preparation of the Hole. Because grout contains fine aggregate, a larger diameter hole is required for grouted anchors than for adhesive anchors. Manufacturer’s instructions should be followed for the hole diameter. The hole should be drilled with a rotary hammer. Using a core drill will produce smooth walls in the hole, reducing the mechanical interlock between grout and wall surface.

The holes should be cleaned using the recommendations for adhesive anchors (see Section A.1.2). After the holes are cleaned, they should be flooded with water 24 hours prior to anchor installation. This procedure reduces water loss from the grout into the surrounding concrete, and ensures proper grout hydration.

A.2.3 Grout Preparation. Grout should be prepared following the manufacturer’s directions.

A.2.4 Placement of Anchors. The grout should be poured into the hole until the hole is 1/3 to 1/2 full. The anchor should then be placed in the hole, rotating the anchor to break up any trapped air bubbles.

A.2.5 Curing of Grout. After initial setting of the anchor, wet rags should be placed over the grout surface for 24 hours. Anchors should then be cured under ambient conditions following the manufacturer's directions.

A.3 Expansion and Undercut Anchors

Because expansion and undercut anchors do not involve the mixing and placing of a bonding agent, the placement of the anchors varies significantly from that of adhesive and grouted anchors.

A.3.1 Preparation of the Hole. Holes should be drilled using a rotary hammer. The diameter of the hole should as directed by the manufacturer. Some undercut anchors require the use of a special undercutting bit. Manufacturer's directions should be carefully followed.

Cleaning of the hole is not critical with expansion and undercut anchors. This is because expansion anchors resist pullout by friction, and undercut anchors resist pullout by a combination of friction and bearing.

A.3.2 Placement of Expansion Anchors. Placement of expansion anchors should be in accordance with recommendations of the manufacturer. Most expansion anchors are torque-controlled. Torque controlled anchors are anchors that are expanded by applying a torque to the anchor. For torque controlled anchors, gently drive the anchor into the hole with a rubber mallet. Then use a torque wrench, set according to manufacturer's guidelines, to expand the anchor in the hole.

A.3.3 Placement of Undercut Anchors. Torque-controlled undercut anchors should be placed into their holes and hammered with a special tool supplied by the manufacturer. This creates the undercutting action. The anchor is then expanded using a torque wrench, similar to the method for expansion anchors. Hydraulic-controlled anchors should be placed into the hole and expanded by tension applied to the anchor by a hydraulic ram. As with expansion anchors, the manufacturer's directions should be carefully followed.

APPENDIX B
INSPECTION AND TESTING REQUIREMENTS
FOR RETROFIT ANCHORS

B.1 Introduction

Field inspection of retrofit anchors is necessary to assure adequate performance. Field inspection should include observation of installation and testing of installed anchors for strength. In addition to field testing, laboratory tests will be necessary to ascertain certain properties of a specific anchor system.

B.2 Laboratory Tests

B.2.1 Adhesive Anchor Systems. In order to properly design adhesive anchors, the maximum bond strength and the λ' value of the adhesive must be determined.

The maximum bond strength is determined by installation and testing of partially bonded anchors with a bonded length of 2 inches. λ' for the adhesive is determined by installation and testing of partially bonded anchors with bonded lengths of 2, 3, and 4 inches.

In order to determine the maximum bond strength and λ' , five anchors should be installed at an embedment length of 4 inches, three at an embedment of 5 inches, and three at an embedment of 6 inches. Installation should be in accordance with Appendix A. The top 2 inches of embedment should be debonded by coating the appropriate section on the anchor and the inside of the hole with a debonding agent such as silicone sealant or duct tape. The anchors should be allowed to cure 24 hours or as recommended by the manufacturer. After curing, the anchors should be tested in tension.

To test the anchors, a center hole ram and hand pump can be used. The ram should be placed on a frame which moves the reaction away from the anchor, to avoid any effect of confinement. Fig. B.1 shows a schematic of an acceptable loading system. Each anchor should be loaded in tension until the ultimate load, P_{eq} , is reached. For each embedment depth, if the result of any one test varies more than 15% from the average, two additional tests shall be performed and the five tests averaged. Using the ultimate loads for each of the anchors, λ' and the maximum bond stress, u_0 , can be found.

To find u_0 :

1. For each of the anchors with a 2 inch bonded length find u_0 : $u_0 = P_{exp}/2\pi d_h$
2. Average all the values for u_0 found above to get u_0 .

To find λ' :

1. Assume a value of λ' - usually between 0.1 and 0.25.
2. For all of the tests, find P_{eq} with this equation:

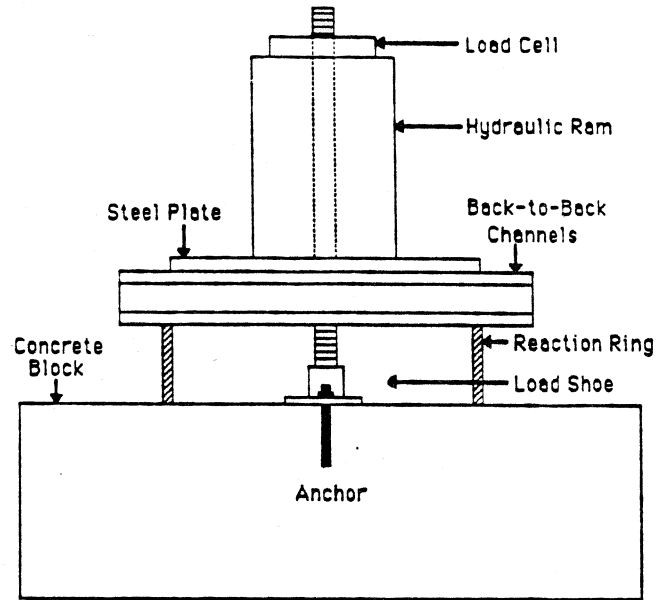


Figure B.1 Schematic Drawing of Loading System

$$P_{eq} = \frac{\pi u_o d_h^{1.5}}{\lambda'} \tanh \left[\frac{\lambda' (l_e - 2)}{\sqrt{d_h}} \right]$$

where:

l_e = embedment length of the anchor

d_h = the diameter of the hole

3. For each test, find the square error, ϵ .

$$\epsilon = (P_{exp} - P_{eq})^2$$

where:

P_{exp} = the experimentally determined capacity of the anchor

4. Sum the values of ϵ for each test in the series. Iterate on λ' to minimize the summation of the square error.

B.3 Field Testing

B.3.1 Observation of Installation. When retrofit anchors are used, a field inspector will observe the installation procedures used. The field inspector should verify that installation procedures adhere to the guidelines in Appendix A. In addition to observation of installation procedure, the inspector should verify appropriate storage and handling of the anchor systems.

For adhesive anchors, the inspector will ascertain that the adhesive is properly stored. In the instructions accompanying each adhesive the manufacturer should set limits for storage temperature and shelf life. These limits should be strictly followed.

For mechanical anchors, the inspector should determine that the anchors are not stored in an environment which might promote corrosion prior to installation.

B.3.2 Load Testing of Anchors. To assure proper installation and design, several anchors in each installation shall be loaded to 80% of minimum specified yield. Three of the first 10 anchors installed, and 10% of the remaining anchors, shall be tested. To test the anchors, a center hole ram and hand pump can be used. The ram shall be placed on a frame which moves the reaction away from the anchor, to avoid any effect of confinement. Fig. B.1 shows a schematic of an acceptable loading system. Any anchor failing to reach 80% of minimum specified yield should be removed and replaced.

An additional criterion must be applied to expansion anchors. If any expansion anchor exhibits excessive slip (greater than 5% of the embedded length) at a tensile load less than or equal to 80% of minimum specified yield, that anchor should be removed and replaced. Slip can be measured by placing a mark on the anchor at the surface of the concrete and measuring displacement with a tape measure.

REFERENCES

The design requirements presented in this guide are based primarily on the following documents:

1. Collins, D. M., Cook, R.A. and Klingner, R. E., "Load- Deflection Behavior of Cast-in-Place and Retrofit Concrete Anchors Subjected to Static, Fatigue, and Impact Tensile Loads," *Report No. 1126-1*, Center for Transportation Research, University of Texas at Austin, February 1989.
2. Doerr, G. T., Cook, R.A. and Klingner, R. E., "Adhesive Anchors: Behavior and Spacing Requirements," *Report No. 1126-2*, Center for Transportation Research, University of Texas at Austin, March 1989.
3. Cook, R. A. and Klingner, R. E., "Behavior and Design of Ductile Multiple-Anchor Steel-to-Concrete Connections," *Report No. 1126-3*, Center for Transportation Research, University of Texas at Austin, March 1989.
4. ACI Committee 349, *Code Requirements for Nuclear Safety Related Structures (ACI 349-85)*, American Concrete Institute, Detroit, 1985.
5. ACI Committee 322, *Building Code Requirements for Structural Plain Concrete (ACI 322-72)*, American Concrete Institute, Detroit, 1972.
6. "General Anchorage to Concrete," *TVA Civil Design Standard No. DS-C1.7.1*, Tennessee Valley Authority, Knoxville, 1984.
7. *Manual of Steel Construction*, 8th Edition, American Institute of Steel Construction, Chicago, Ill., 1980.
8. *Manual of Steel Construction, Load and Resistance Factor Design*, 1st Edition, American Institute of Steel Construction, Chicago, Ill., 1986.

The following gives the specific references from which each section of this *Guide* was derived. If a Section does not appear in the following, it either contains no design provisions or contains design provisions which are unique to this *Guide*. For those Sections which contain design provisions unique to this *Guide* the provisions are justified in the *Guide* and are believed to be conservative.

SECTION	REFERENCE
1.2 General Design Philosophy	3,4,6
1.3 General Design Requirements	3,4
1.4 Strength Reduction Factors	2-5,7,8
2.3 Nominal Strength of Steel for Single Anchors	3
2.4 Nominal Strength of Embedment for Single Anchors	1,2,4,6
2.5 Ductile and Non-Ductile Anchors	1,3
2.6 Special Case: Quasi-Ductile Anchors	4,6

2.7	Anchor Head Requirements	4,6
3.1	General Requirements	3
3.2	Design Philosophy	3
3.3	Design Procedure for Moment	3
3.4	Design Procedure for Moment with Axial Compression	3
3.5	Design Procedure for Moment with Axial Tension	3
3.6	Design Procedure for Shear	3,4,6
4.3	Nominal Strength of Steel for Ductile Connection Embedment Design	3,4
4.4	Nominal Strength of Embedment for Connections	2,4,6
4.5	Calculation of Projected Area for Overlapping Failure Cones	3
6.2	Design Philosophy	3
6.3	Baseplate Design Procedure for Flexure	3,8
6.4	Baseplate Design Procedure for Shear	8
6.4	Baseplate Design Procedure for Concrete Bearing	4,8
6.5	Baseplate Design Procedure for Steel Bearing	8
6.6	Baseplate Anchor Hole Requirements	3,8