TENSILE CAPACITY OF SINGLE ANCHOR IN CONCRETE

EVALUATION OF EXISTING THEORY

ON AN LRFD BASIS

by

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1.0 INTRODUCTION

1.1 General

One of today's most vexing problems in the design of anchorage to concrete is how to predict the tensile capacity of anchors (this research is limited to cast in place, expansion and undercut anchors, but not to adhesive anchors), as governed by failure of the concrete. Many sets of design recommendations exist. Some come primarily from a cone-based theory, long popular in the United States. Others come from the "Kappa theory," developed over the past decade in Europe. Both theories are reasonably consistent with test data.

To determine which theory is most appropriate, the available data base must be carefully examined. Resolution of this issue will permit a rational, consistent evaluation of the capacity of existing anchors, will unify the code development process in the United States, and will stimulate the increased use of anchors in the U.S. and abroad.

1.2 Scope and Objectives

The overall objective of this research is to evaluate the accuracy and suitability for design of each theory for predicting anchor capacity as governed by concrete failure. That objective will be carried out as follows:

1) Approximately 800 data points, consisting of data previously surveyed by Fuchs and Breen [2] and from personal correspondence with Cannon [3] and Eligehausen [4], are available from tests on single anchors far from a free edge and far from other anchors, and failing by formation of a concrete cone. Using common definitions and nomenclature for all variables and material properties, those data will be placed in two parallel data bases, one in terms of U.S. units and concrete cylinder strengths, the other in terms of SI units and concrete cube strengths. Anchor types include cast in place anchor bolts, undercut anchors, and expansion anchors. No adhesive anchors were included.

2) All data will be plotted against three existing theories: the 45° cone theory of <u>ACI 349-85</u>; a variable-angle cone theory; and the Kappa theory (exponent of 1.5). The plots will show concrete capacity, normalized by $\sqrt{f_c}$, as a function of embedment depth.

3) For different ranges of embedment depths, observed data will be compared against those existing theories in terms of square error.

4) For different ranges of embedment depths, an evaluation will be conducted of the probability of failure under known loads, and also the probability of concrete failure under unlimited loads, of anchors designed according to each theory.

5) Based on those comparisons, each theory will be evaluated with respect to accuracy and design suitability.

2.0 BACKGROUND

2.1 General

To design anchors subjected to tension, either acting alone or in combination with other actions, it is necessary to predict the load at which a single tensile anchor will fail, as governed by failure of the concrete. Two principal theories are available: the cone theory and the so-called "Kappa" theory. The former is the basis for one widely used U.S. anchor design code [1]; the latter is described in recent European technical literature [5,6,7].

2.2 Cone Theory

Most U.S. design recommendations are based on the cone theory [1,8], named after the roughly conical shape of the piece of concrete which pulls out in this kind of failure. Although this theory is often used assuming a cone angle of 45°, it is applicable to arbitrary cone angles.

Calculations by the cone theory assume a uniform nominal stress f_t on the entire surface of a truncated cone. The nominal tensile strength f_t, usually taken as $4\sqrt{f'_c}$ in psi units, is about two-thirds the typical actual tensile strength of $6\sqrt{f_c}$. The cone theory is therefore usually justified physically in terms of a tensile stress distribution which varies from a maximum of $6\sqrt{f_c}$ at the anchor head to zero at the surface. However, it can also be justified physically in terms of a uniform stress of $6\sqrt{f_c}$ acting over the uncracked portion of a slowly propagating conical failure surface. When this theory is used for design, the concrete strength f_c is usually taken as the minimum specified strength f_c'.

The cone theory has some arguments in its favor: first, the final failure surface is approximately conical; and second, the theory is conceptually simple. The principal objections to the cone theory are that it does not account directly for experimental evidence from the U.S. and Europe which clearly shows that failure does not occur simultaneously over the entire surface, and that the cone angle is not constant along the surface of the failed piece of concrete [5,6,7,9,10]. While cone theories can include both phenomena, they do not always do so.

2.2.1 Constant-Angle Cone Theory:

If the cone angle is assumed constant with embedment depth, the cone theory predicts that anchor capacity will increase as a function of the embedment depth squared (that is, raised to the power 2). Constant-angle cone theories are exemplified by the provisions of Appendix B of ACI 349-85 [1].

2.2.2 Variable-Angle Cone Theory:

Experimental observations confirm that the average cone angle decreases (that is, the cone becomes flatter) as the embedment depth decreases. Within each cone, the cone angle also decreases as the fracture propagates toward the surface. TVA design recommendations [11] have addressed this by prescribing a cone angle which is equal to 45° for embedment depths equal to or greater than 5 inches, and which decreases linearly to 28° for embedment depths of zero. If a flatter cone angle is assumed for shallower embedments, calculated anchor capacity will increase more slowly with embedment depth than the constant-angle cone theory would predict. Because the variable-angle cone theory modifies the original cone theory by a coefficient which decreases with increasing embedment, it simulates a constant-coefficient equation in which embedment depth is raised to a power less than 2.

2.3 Kappa Theory

The Kappa theory [5,6,7], based on a combination of empirical evidence plus some fracture mechanics, states that the anchor capacity, as governed by concrete failure, varies as a function of the embedment depth to the power 1.5.

The Kappa theory has several arguments in its favor:

a) Reduced test data from Europe, obtained from tests in which all variables except embedment depth were maintained constant, suggest that capacity prediction formulas with an exponent of 1.5 are preferable to those with an exponent of 2 [12].

b) It has been proposed that fracture mechanics may provide a theoretical basis for capacity prediction formulas with an exponent of 1.5 [10].

The principal objection to the Kappa theory has been that most European data have not previously been available in their original form. This difficulty has been overcome by the recent work of Eligehausen, Fuchs and Breen in this area [2]. It has also been argued that fracture mechanics theory does not provide unequivocal support for the exponent of 1.5, nor for the precise form of the Kappa theory equation.

3.0 REVIEW OF AVAILABLE ANCHOR FAILURE DATA

3.1 General Description of Available Anchor Failure Data

3.1.1 Anchor Failure Data Obtained from Fuchs and Breen:

Data on concrete failures was obtained from a data base previously compiled by Fuchs and Breen [2], using results of tests on anchor bolts, headed studs, undercut anchors, and expansion anchors. That data base is presented in its original form in Appendix A of this thesis. In Appendix A, each set of data is identified by a corresponding reference number. The references are listed at the end of Appendix A.

Each line of the data base describes a single test result in terms of the following information:

1) Syst.: test number as defined by Fuchs and Breen
2) d: diameter of the anchor
3) do: diameter of the anchor head for headed anchors or hole diameter for non-headed anchors
4) hef: effective embedment depth (measured to bearing surface)

6)	fy:	yield strength of the steel
7)	ft:	tensile strength of the steel
8)	fcc200:	compressive strength of 200 mm concrete cubes
9)	fct:	tensile strength of the concrete
10)	dagg	diameter of the aggregate in the concrete
11)	c1:	edge distance
12)	s1:	distance to nearest anchor (direction 1)
13)	s2:	distance to nearest anchor (direction 2)
14)	Nu:	actual failure load
15)	Lit.:	reference number

The effective embedment (measured to the bearing surface of the anchor) is illustrated in Fig. 1.

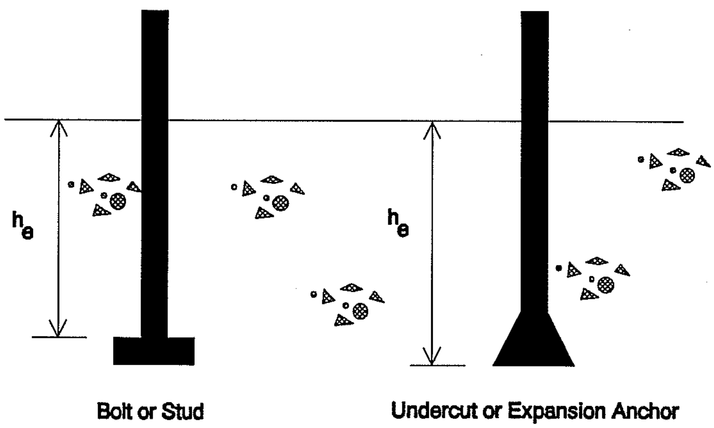

Figure 1: Definition of Effective Embedment Depth for Anchors

The original concrete data base contained the results of several thousand tests. As discussed in Section 3.2.4, many of these were tests involving groups and edge effects and so were excluded from further consideration. A total of 801 data points remained. Because most of the concrete failures had been obtained from European tests, the original concrete data base had been prepared in SI units. Concrete strength was expressed in terms of cube strength.

3.1.2 Anchor Failure Data Obtained from Klingner et al:

Data on steel failures was obtained from data previously published by Klingner et al [8] and Collins [6] , using results of tests on anchor bolts and headed studs. The data base of Klingner et al [8] is presented in its original form in Appendix B of this thesis. In Appendix B, each set of data is identified by a corresponding reference number. The references are listed at the end of Appendix B.

Each line of the data base describes a single test result in terms of the following information:

1) test number as defined in Reference 8
2) Reference number as listed in Reference 8
3) test number as defined in original reference
4) type of anchor
5) diameter of anchor

6) length of anchor
7) effective embedment depth of anchor
8) diameter of the anchor head
9) compressive strength of the concrete
10) type of concrete (normal or lightweight aggregate)
11) failure load
12) failure mode (steel or concrete)
13) remarks

These data refer to single-anchor tests only. A total of 90 steel failure points were used. Because most of the steel failure data had been obtained from U.S. tests, the original steel data base had been prepared in U.S. units. Concrete strength was expressed in terms of cylinder strength.

3.2 Operations Carried Out on Original Data Bases

3.2.1 Placement of Data in Spreadsheet-Type Format:

The original data bases of Appendices A and B were placed in spreadsheet form using SuperCalc 5.0©. They can be readily converted for use with Fortran or BASIC programs, other spreadsheets such as LOTUS*123©, and most common graphics packages.

3.2.2 Conversion of SI to U.S. Units and Vice-Versa:

The original data base for concrete failures was obtained in SI units. In contrast, the original data base for steel failures was obtained in U.S. units. Using the conversion factors given below, all original data were manipulated to obtain two parallel data bases--one in U.S. units and one in SI units:

1 in	=	25.4 mm
1 lb	=	4.448 N
1 psi	=	0.006895 N/mm^2
f_{cc} (cube)	=	1.18 f_c (cylinder)

After these conversions had been made, the predicted failure loads were computed separately for each data base, and were normalized for differences in concrete strengths.

Although the normalized predicted failure loads were computed separately for U.S. and SI data bases, a direct conversion between the two is also possible. Consider Test #1 from Appendices C and D respectively. The <u>ACI 349-85</u> equation predicts a normalized failure load of 803.6 $\sqrt{lb \cdot in}$. In Appendix D, the same data point is given as 39,606 $\sqrt{N \cdot mm}$. A direct conversion can be made as follows:

$$803.6\sqrt{lb\cdot in}\ x\ 25.4\ \frac{mm}{in}\ x\ \sqrt{\frac{4.448N}{1lb}}\ x\ \sqrt{\frac{1}{1.18}} = 39630\sqrt{N\cdot mm}$$

The slight difference in converted values occurs because the coefficient in the SI equation is rounded to two significant figures.

U.S. practice bases the compressive strength of concrete on the strength of a 6- by 12-inch cylinder, while European practice uses a 200-mm cube. Because of its shape, a concrete cube normally has a central region subjected to biaxial lateral confinement due to platen restraint. If cubes and cylinders are cast from a single batch of concrete and cured identically, this causes the cubes to have a higher apparent strength than the cylinders. The cube strength is about 1.18 times the corresponding 6 x 12 cylinder strength [13]. This value is consistent with conversion factors used by Fuchs and Breen.

3.2.3 Creation of Parallel U.S. and SI Data Bases:

After converting units and concrete strengths, the original data bases were placed in spreadsheet form. Two parallel sets of data bases were created, one in U.S. units, the other in SI units. Each line of the SI data base contains the same information as the corresponding line of the U.S. data base. Information presented here can be expressed and evaluated in either set of units and concrete compressive strength references.

3.2.4 Selection of Data to be Excluded:

Some data in the original concrete data files were excluded from further consideration, based on the following criteria:

1) Test data from multiple anchors were excluded. It would have been possible to include the multiple anchor data from those tests in which the anchors were placed sufficiently far apart so as not to interfere with one another. However, because sufficient single anchor data were available, inclusion of the multiple-anchor data was not considered necessary.

2) Test data from single anchors located close to a free edge were excluded. Anchors placed close to a free edge can fail prematurely by creation of a partial cone [5,8], or by side blowout [5,8,14,15]. The edge distance within which a partial cone forms is far greater than that at which side blowout is a problem. Therefore, by excluding all data which might have influenced by the formation of a partial cone, the influence of side blowout was also excluded.

The shallowest cone angle commonly observed is about 28°. Consistent with this, it has been suggested [7] that anchors will not be affected by a free edge unless they are placed closer to the free edge than ($h_e/\tan 28^o$), or 1.9 h_e. Based on this criterion, data were excluded in all cases in which the edge distance was less than twice the effective embedment depth.

3.2.5 Normalization of Data by Concrete Strength:

The available data were obtained from tests involving many different values of concrete strength. All three theories examined here predict that anchor capacity as governed by concrete failure will be proportional to the square root of the concrete compressive strength. To permit the data to be plotted on a single graph as a function of embedment depth, actual and predicted failure loads were normalized by the square root of the concrete compressive strength.

3.2.6 Possible Non-Compliance of Data with <u>ACI 349-85</u> Requirements:

As discussed in detail in Section 7.2, <u>ACI 349-85</u> has requirements for the minimum bearing area of the anchor head. The concrete data base was not checked for compliance with those requirements.

3.3 Plots of Observed Capacity versus Embedment Depth

Figures 2 and 3 show (in U.S. and SI units respectively) all data for concrete failures, normalized by $\sqrt{f_c}$, and plotted as a function of embedment depth.

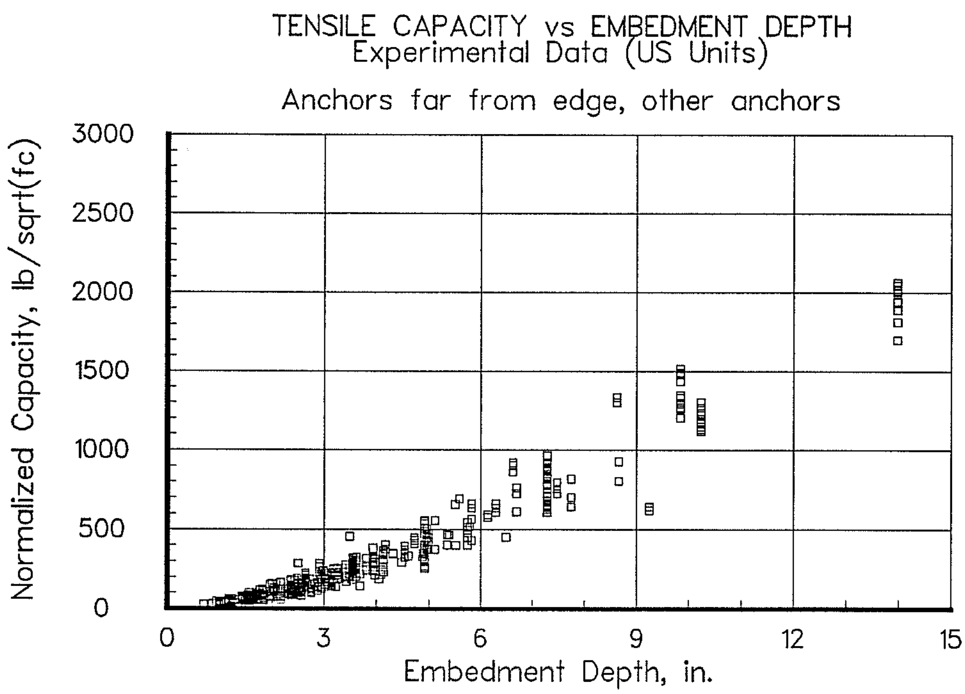

Figure 2: Normalized Anchor Capacities as Governed by Concrete Failure, U.S. Units

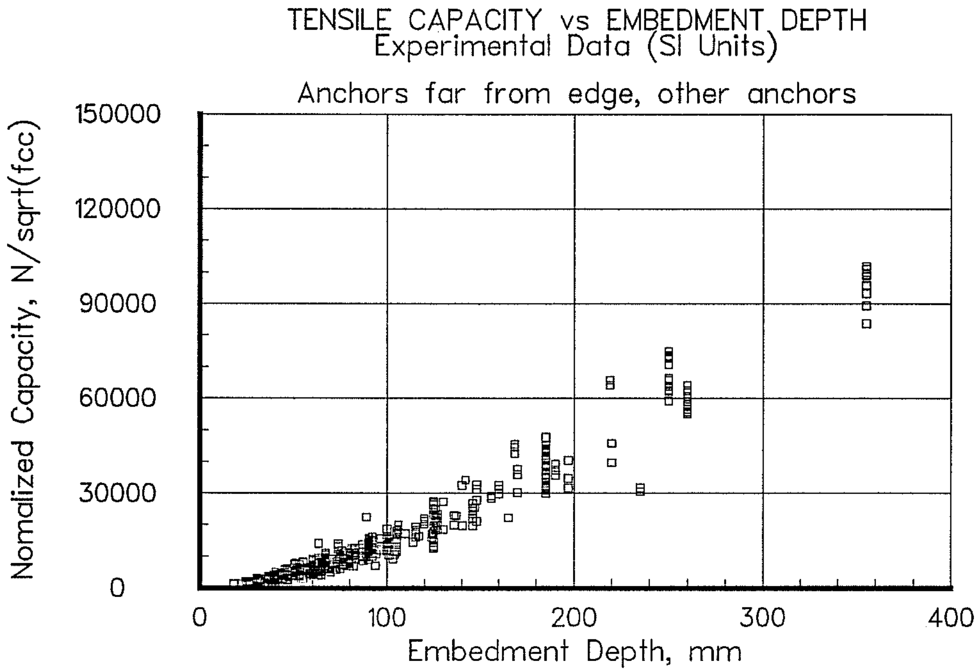

Figure 3: Normalized Anchor Capacity as Governed by Concrete Failure, SI Units

4.0 DESCRIPTIONS OF PRINCIPAL EXISTING THEORIES

4.1 General

In this section the three principal existing theories introduced above are described mathematically in U.S. and SI units. In evaluating the ACI 349-85 theory and the variable-angle cone theory for predicting concrete capacity, the head diameter d_h was taken to be the actual diameter of the anchor. This is believed consistent with the latest deliberations of ACI Committee 349.

Sample calculations will be given for computation of anchor capacity according to each theory. When capacity prediction theories are used for design, they are expressed in terms of specified concrete compressive strength (f'_c in U.S. practice). When capacity prediction theories are compared with test results, the actual concrete compressive strength is used (denoted by f_c or f_{cc} for U.S. and European data, respectively). So that the equations will remain consistent with their original sources, they are written using the notation of f'_c. In comparing theories, no ϕ (understrength) factors were used with any of the equations.

4.2 Description of the Failure Theory of ACI 349-85, Appendix B [1]

The nominal cone capacity of the concrete is based on a maximum tensile stress strength of $4\sqrt{f'_c}$ (psi units), acting on the projected area of a 45° stress cone radiating toward the attachment from the bearing edge of the anchor (B.5.1.1):

$$4\,\pi\sqrt{f'_c}\,h_e\,(h_e + d_h)$$

where h_e = embedment length, measured from free surface to the bearing surface of the anchor head

d_h = diameter of anchor head (taken as anchor diameter)

4.2.1 Equations Used with ACI 349-85 and U.S. Units:

When using U.S. units (pounds, inches, and concrete cylinder strength), the concrete cone capacity is given by:

$$4\,\pi\sqrt{f'_c}\,h_e\,(h_e + d_h)$$

Considering the first data point in Appendix C, the variables in the above equation are f'_c = 5297 lb/in^2, h_e = 7.622 in, and d_h = 0.768 in. The predicted concrete capacity is 58.4 kips.

4.2.2 Equations Used with ACI 349-85 and SI Units:

When using SI units (kN, mm, and concrete cube strength), the concrete cone capacity is given by:

$$0.96 \sqrt{f_{cc}}\, h_e \,(h_e + d_h)$$

Considering the first data point in Appendix D, the variables in the above equation are f_{cc} = 43.1 N/mm^2, h_e = 193.6 mm, and d_h = 19.5 mm. The predicted concrete capacity is 260.0 kN.

4.3 Description of the Variable-Angle Cone Failure Theory

The variable-angle cone failure theory is identical to that of Appendix B of ACI 349-85, except for the way in which the available concrete cone capacity is calculated [8], the variable-angle cone theory used here is based on one original developed by the Tennessee Valley Authority [8]. The design pullout strength of the concrete is based on a maximum tensile strength of $4\sqrt{f'_c}$ (psi units), acting on the projected area of a variable-angle stress cone radiating toward the attachment from the bearing edge of the anchor:

$$4\pi\sqrt{f'_c}\left(\frac{h_e}{\tan\theta}\right)\left[\left(\frac{h_e}{\tan\theta}\right)+d_h\right]$$

where h_e = embedment length, measured from free surface to the bearing surface of the anchor head

d_h = diameter of anchor head (taken as anchor diameter)

θ = cone angle, measured from a plane perpendicular to the anchor axis

The cone angle varies as a function of the embedment depth [8,11]:

$$\theta = 45^o \qquad \text{for } h_e \geq 5 \text{ in.} \quad (127.0 \text{ mm})$$

$$\theta = 28^o + (3.4\,h_e)^o \quad \text{for } h_e < 5 \text{ in.}$$

$$(\theta = 28^o + (0.13386\,h_e)^o \quad \text{for } h_e < 127.4 \text{ mm})$$

4.3.1 Equations Used with Variable-Angle Cone Theory and U.S. Units:

When using U.S. units (pounds, inches, and concrete cylinder strength), the concrete cone capacity is given by:

$$4\,\pi\,\sqrt{f'_c}\left(\frac{h_e}{\tan\theta}\right)\left[\left(\frac{h_e}{\tan\theta}\right)+d_h\right]$$

Considering the first data point in Appendix C, the variables in the above equation are f'_c = 5297 lb/in^2, h_e= 7.622 in, and d_h = 0.768 in. The predicted concrete capacity is 58.4 kips.

4.3.2 Equations Used with Variable-Angle Cone Theory and SI Units:

When using SI units (kN, mm, and concrete cube strength), the concrete cone capacity is given by:

$$0.96\,\sqrt{f_{cc}}\left(\frac{h_e}{\tan\theta}\right)\left[\left(\frac{h_e}{\tan\theta}\right)+d_h\right]$$

considering the first data point in Appendix D, the variables in the above equation are f_{cc} = 43.1 N/mm^2, h_e = 193.6 mm, and d_h = 19.5 mm. The predicted concrete capacity is 260.0 kN.

4.4 Description of the Kappa Theory

The Kappa theory predicts that the concrete capacity will increase with embedment length raised to the power 1.5 [6,7,12]. In addition, fracture mechanics has been used to provide some theoretical basis for capacity prediction formula with an exponent of 1.5. In SI units, the Kappa theory is expressed as shown below [5].

15.5 $h_e^{1.5}$ *for headed studs*

13.5 $h_e^{1.5}$ *for threaded anchors (expansion, undercut)*

where h_e = embedment length, measured from free surface to the bearing surface of the anchor head

4.4.1 Equations Used with Kappa Theory and U.S. Units:

When using U.S. units (pounds, inches, and concrete cylinder strength), the concrete cone capacity is given by:

40.24 $h_e^{1.5}$ *for headed studs*

35.05 $h_e^{1.5}$ *for threaded anchors (expansion, undercut)*

Considering the first data point in Appendix C, the variables in the above equation are f'_c = 5297 lb/in^2, h_e = 7.622 in, and d_h = 0.768 in. The predicted concrete capacity is 61.6 kips.

4.4.2 Equations Used with Kappa Theory and SI Units:

When using SI units (kN, mm, and concrete cube strength), the concrete cone capacity is given by:

$15.5\ h_e^{1.5}$ *for headed studs*

$13.5\ h_e^{1.5}$ *for threaded anchors (expansion, undercut)*

Considering the first data point in Appendix D, the variables in the above equation are f_{cc} = 43.1 N/mm^2, h_e = 193.6 mm, and d_h = 19.5 mm. The predicted concrete capacity is 274.1 kN.

5.0 COMPARISON OF EXISTING THEORIES WITH AVAILABLE DATA

5.1 General

The equations given in Section 4 for each of the three theories (U.S. and SI units) were normalized by dividing by $\sqrt{f'_c}$ or $\sqrt{f_{cc}}$ respectively. The results were graphed, and were compared with the observed failure data for each theory, in terms of U.S. and SI units.

Figures 4 through 9 include concrete data points, denoted by boxes. Because the data base contains only a small number of concrete failures at deep embedments, it had been suggested that the data base in this range be augmented by some steel failure data. Individual data points representing steel failure could be regarded as lower bounds to the concrete capacity for those tests, since the concrete capacity must have been at least as high as the load at which steel failure was observed. However, because few steel failure points were available, and because they were very conservative, it was not helpful to include them, and this suggestion was not pursued further.

5.2 Comparison of ACI 349-85, Appendix B with Concrete Failure Data Base

Nominal capacities according to ACI 349-85, Appendix B, are plotted against the data base of concrete failures. The results are shown in Figures 4 and 5 for U.S and SI units, respectively. Those figures show that the assumption of a 45 degree cone is conservative for embedments less than about 8 inches (203 mm), but increasingly unconservative for deeper embedments.

5.3 Comparison of Variable-Angle Cone Angle Theory with Concrete Failure Data Base

Nominal capacities according to the variable-angle cone theory, are plotted against the data base of concrete failures. The results are shown in Figures 6 and 7 for U.S and SI units, respectively. Those figures show that the assumption of a variable cone is reasonable for embedments less than about 8 inches (203 mm), however, the use of a 45 degree cone at deeper embedment gives the same unconsevatism noted above.

5.4 Comparison of Kappa Theory with Concrete Failure Data Base

Nominal capacities according to Kappa theory are plotted against the data base of concrete failures. The results are shown in Figures 8 and 9 for U.S and SI units, respectively. Those figures show that the predictions of the Kappa theory tend to follow the observed data more closely than do either the theory of ACI 349-85 Appendix B, or the variable-angle cone theory.

5.5 Evaluation of Error Associated with Each Theory

The error associated with each theory was evaluated by computing the square root of the sum of the squares of the errors between the predicted and observed values, for each test in the data base. The results are expressed in Tables 1 and 2, in terms of U.S. units $(\sqrt{lb} \cdot in)$ and SI units $(\sqrt{N} \cdot mm)$, respectively. Results are also shown in the form of bar charts in Figures 10 and 11.

5.6 Discussion of Error Evaluation by Sum of Squares Method

Tables 1 and 2 (and Figs. 10 and 11) show that for most embedment depths, the Kappa theory has a square error lower than that of either ACI 349-85 or the

Table 1: Comparison of Error Using Each Theory (Square Root of Sum of Square Errors), U.S. Units

Theory / **Embedment Depth in**	ACI 349-85	Variable-Angle Cone	Kappa
0.01-1.0	18.8	8.1	6.5
1.01-2.0	32.5	20.9	14.8
2.01-3.0	48.7	40.9	28.8
3.01-4.0	65.1	41.8	38.0
4.01-5.0	79.6	72.9	69.5
5.01-6.0	94.2	94.2	86.6
6.01-8.0	120.0	120.0	122.0
> 8.01	470.0	470.0	203.0

variable-angle cone theories. For embedment depths of 4 inches (101.6 mm) or less the Kappa theory has about half the error of ACI 349-85. For embedments from 4 inches to 8 inches (203.2 mm) the error is about the same. For embedments greater than 8 inches (203.2 mm), the Kappa theory has again about half the error. Errors of all theories increase for embedment depths greater than 6 in. (150 mm). Tables 1 and 2 are consistent with Figures 4 through 9.

However, this method of error analysis does not present a complete picture of the reliability of a given formula. It assigns more weight to data points located far from the values predicted by the equation under consideration. Because each data point does not contribute equally in the error analysis, some distortion is created. A few data points lying far from the curve can have as much effect as a larger number of points

Table 2: Comparison of Error Using Each Theory (Square Root of Sum Square Errors), SI Units

Theory / **Embedment Depth mm**	ACI 349-85	Variable-Angle Cone	Kappa
0.3-25.4	927	400	319
25.7-50.8	1605	1033	728
51.1-76.2	2399	2018	1422
76.5-101.6	3209	2060	1876
101.9-127.0	3924	3595	3426
127.3-152.4	4643	4643	4272
152.7-203.2	5908	5908	5990
> 203.5	23169	23169	10021

close to the curve. Also, this method does not distinguish systematic error from random error. Examination of Figures 4 through 9 shows that the ACI 349-85 cone theory is consistently conservative for low embedments. That fact is not revealed in comparison of square error.

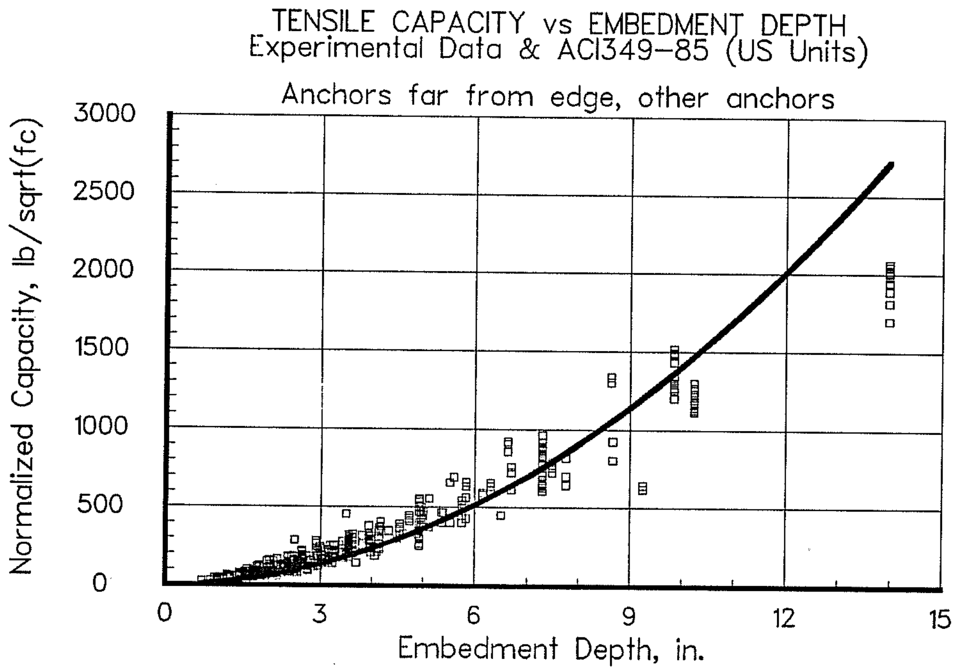

Figure 4: Comparison of Observed and Predicted Concrete Capacities, ACI 349-85, Appendix B, U.S. Units

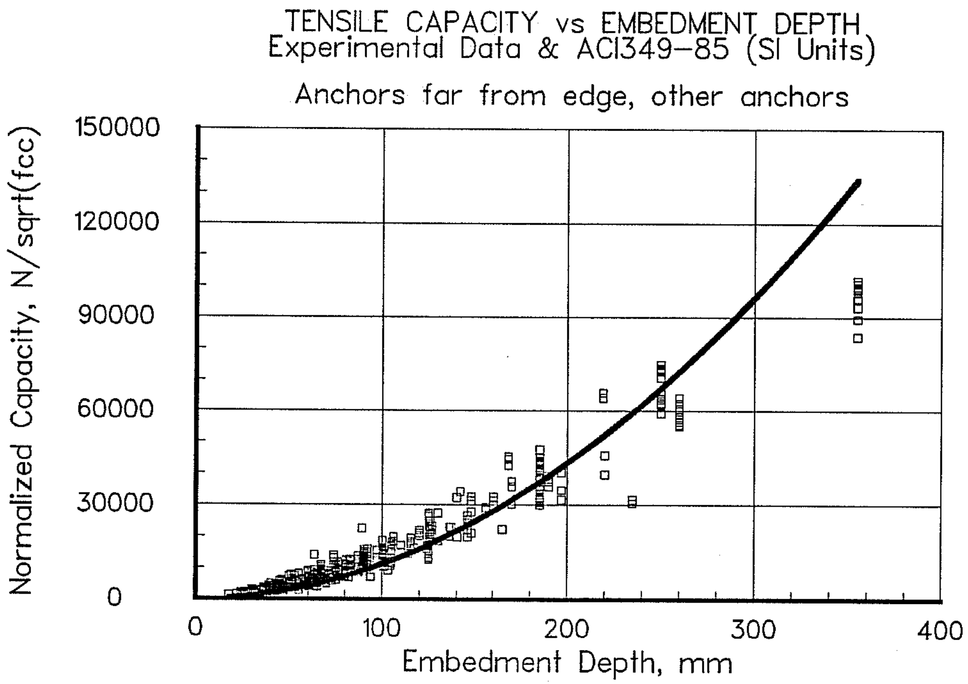

Figure 5: Comparison of Observed and Predicted Concrete Capacities, ACI 349-85, Appendix B, SI Units

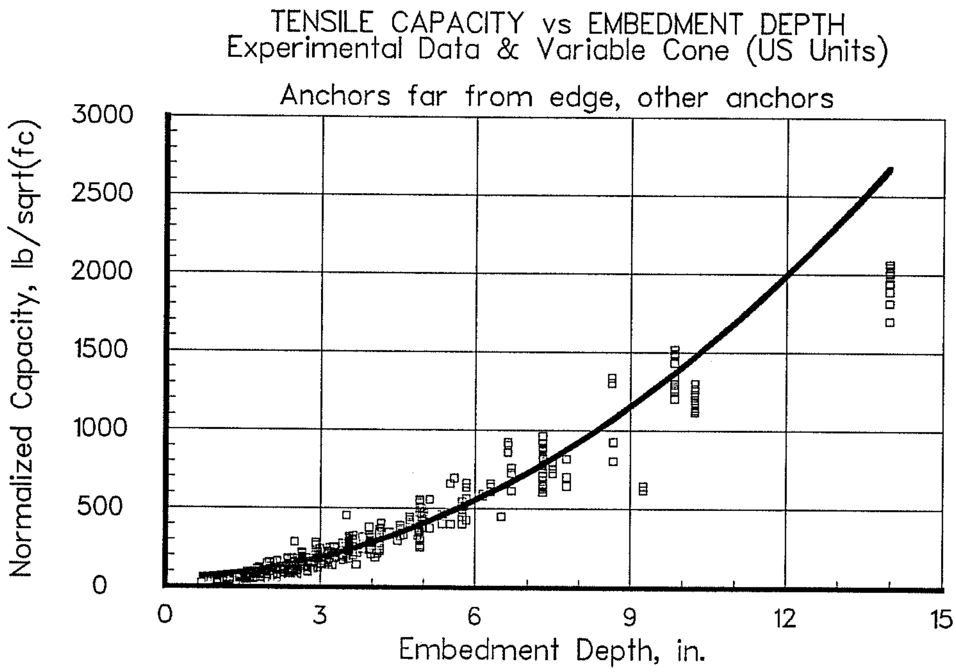

Figure 6: Comparison of Predicted and Observed Concrete Capacities, Variable-Angle Cone, U.S. Units

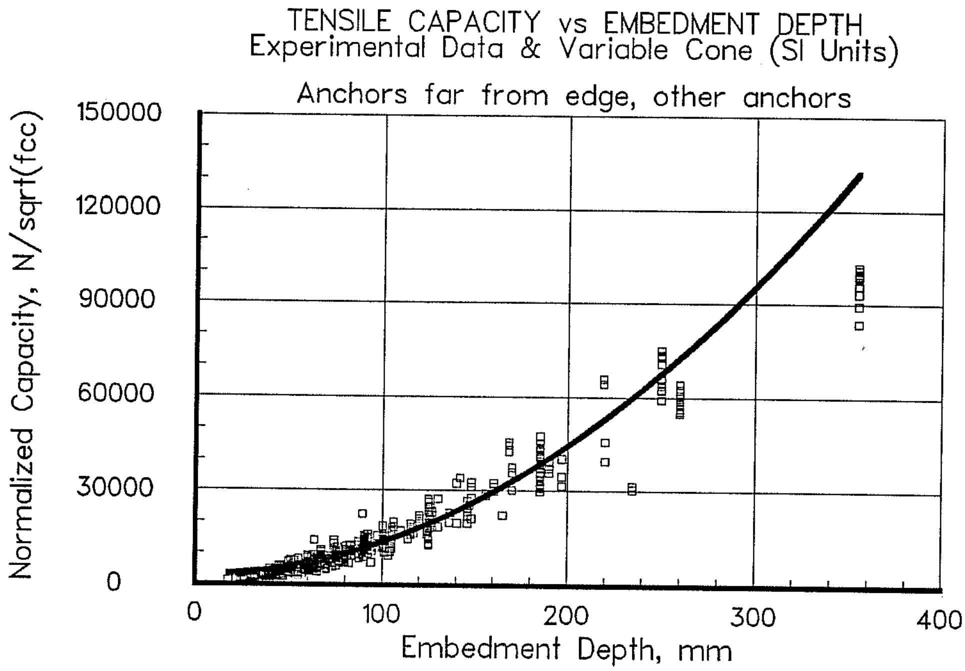

Figure 7: Comparison of Observed and Predicted Concrete Capacities, Variable-Angle Cone, SI Units

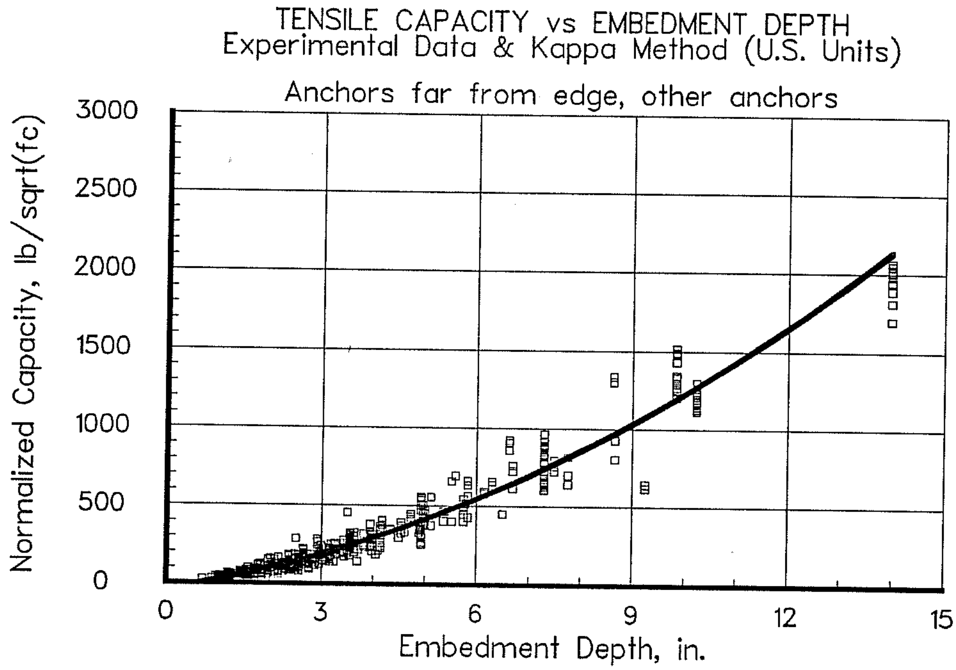

Figure 8: Comparison of Observed and Predicted Concrete Capacities, Kappa Theory, U.S. Units

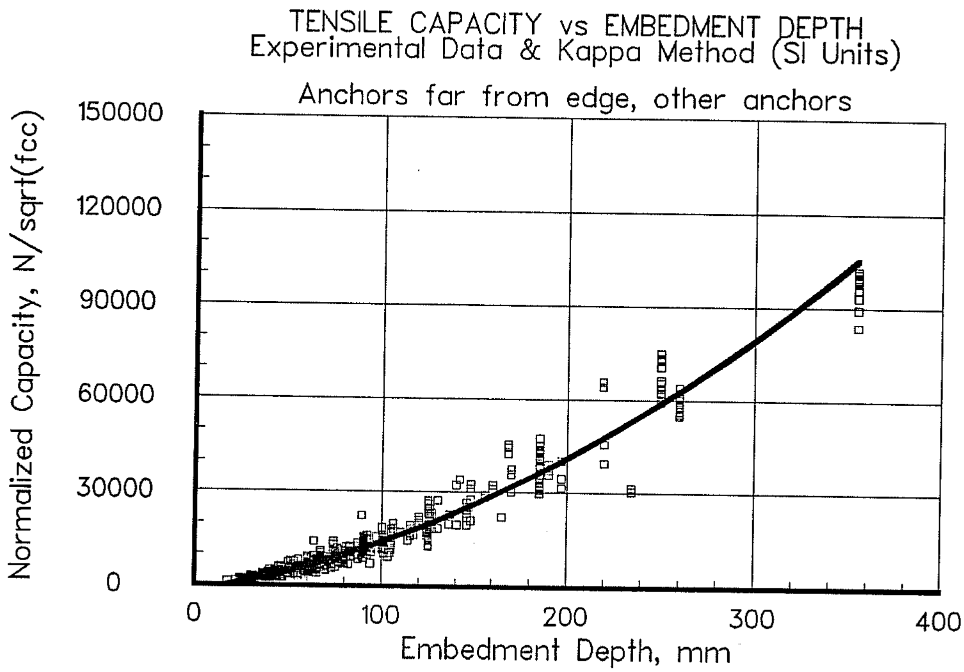

Figure 9: Comparison of Observed and Predicted Concrete Capacities, Kappa Theory, SI Units

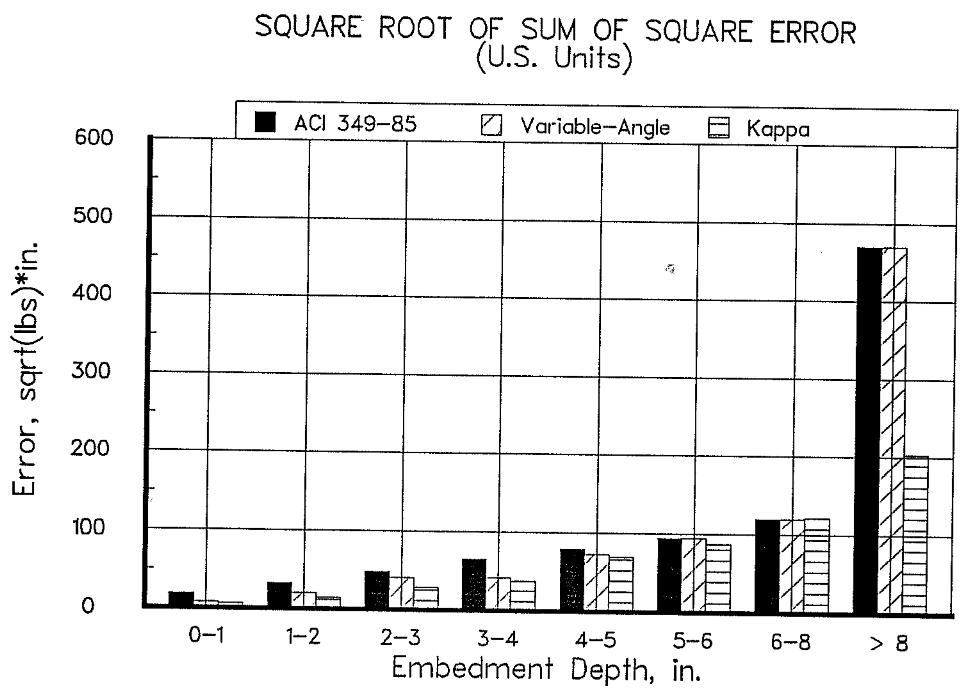

Figure 10: Comparison of Error Using Each Theory (Square Root of Sum of Square Error), U.S. Units

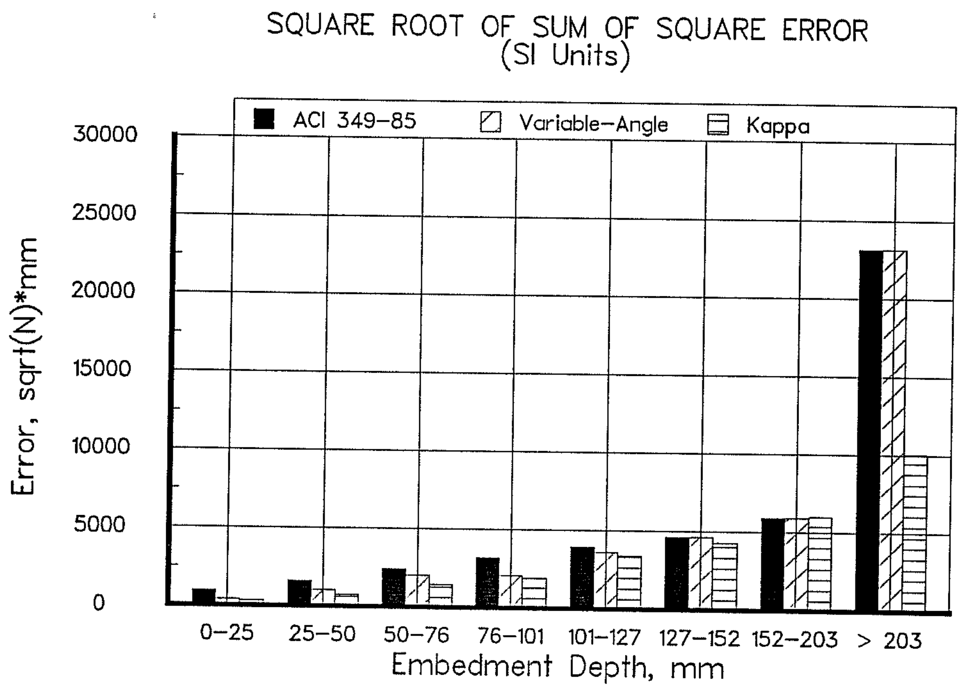

Figure 11: Comparison of Error Using Each Theory (Square Root of Sum of Square Error), SI Units

6.0 LRFD IMPLICATIONS OF EXISTING THEORIES

6.1 General

In this section, the design implications of each of the three existing theories (ACI 349-85, variable-angle cone, and the Kappa theory) are evaluated in terms of the probable behavior of anchors designed according to each theory. This behavior is presented in terms of Load and Resistance Factor Design (LRFD) [16].

The focus of this research is not to compare the design procedures of other codes with that of ACI 349-85. Rather, the main objective is to test three theories with respect to their reliability and suitability for design within the design approach of a single code: ACI 349-85. Because there is little disagreement regarding the behavior of anchors as governed by steel failure, and because this research is focused on comparisons among different theories (ACI 349-85, variable-angle cone and kappa) for predicting anchor capacity as governed by concrete failure, the following procedure is used with each theory:

1) Arbitrary mean factored loads are assumed. The loads are also assumed to be normally distributed, with an arbitrary coefficient of variation.

2) Based on those loads, and using the design provisions of ACI 349-85, Appendix B, the required steel area is computed.

3) Using the results of the steel failure data published by Klingner et al [8] and Collins [9], the corresponding statistical distribution of actual anchor steel strengths (A490 and A193-B7) is computed.

4) Based on the ductile failure provisions of ACI 349-85, Appendix B, the required embedment is computed, corresponding to the required steel area. Three different required embedments are computed, one corresponding to each of the three theories investigated here. A constant understrength factor of 0.65 is used in each case.

5) Based on the relationships derived above for the predictive accuracy of each formula in each range of embedment depths, the probable statistical distribution of actual anchor strengths as governed by concrete is computed, assuming that the anchors were designed by each of the three theories investigated here.

6) Using a Monte Carlo simulation, the probability of failure under known loads is computed, for each theory, for a range of embedment depths. For purposes of this study, failure is defined as occurring whenever the actual load would exceed the

lesser of the actual concrete resistance or the actual steel resistance.

7) Using a Monte Carlo simulation, the probability of concrete failure under unlimited loads is computed, for each theory, for a range of embedment depths. The concept of "unlimited load" is introduced for the following reason: some extreme loads such as strong earthquake, are highly unpredictable in magnitude. Because of this, design loads for extreme earthquake are often based on the structural actions associated with the formation of plastic mechanism, since the formation of such a mechanism sets an effective upper limit on the actions. By "unlimited loads" therefore, this thesis refers to loads that are limited only by the capacity of the weakest element in the anchoring systems. The probability of concrete failure under unlimited loads is equivalent to the probability that steel capacity will exceed concrete capacity. This definition is believed to be conservative.

8) For each concrete failure theory, and for different ranges of embedment depths, graphs were prepared showing the probability of failure (steel or concrete) under known loads, and the probability of concrete failure under unlimited loads.

6.2 Review of Tensile Anchor Design Provisions of ACI 349-85 [1]

The design philosophy of Appendix B of ACI 349-85 is a strength design approach, in which anchors are designed to fail in a ductile manner. Steel is expected to yield over a significant portion of the anchor length before steel fracture, concrete cone failure, or anchor pullout. In the design of an anchor for direct tension, this philosophy leads to the requirement that the concrete pull-out strength must exceed the tensile strength of the anchor. For those cases where ductile failure modes cannot be assured, ACI 349-85 requires that the allowable load carrying capacity be reduced.

ACI 349-85 applies to cast-in-place, expansion, and undercut anchors, but not to adhesive anchors. Its specific provisions with regard to the design of tensile anchors are discussed below; equation and section numbers refer to that document.

Design for tensile load involves specifying sufficient anchor area so that the tensile capacity of the anchor, reduced by a capacity reduction factor, will exceed the factored design tension acting on the anchor (B.6.2):

$$\text{Factored Design Tension} \leq \phi\, A_s\, f_y$$

but

$$\text{Factored Design Tension} \leq A_s\,(0.8\, f_{ut})$$

where A_s = tensile stress area of anchor steel

ϕ = 0.9 (capacity reduction factor)

f_y = specified minimum yield strength of anchor steel

f_{ut} = specified ultimate tensile strength of anchor steel

To prevent local bearing failure of the concrete, the bearing area of the anchor head (excluding the area of the shank) must be at least 1.5 times the area of the shank, and the thickness of the anchor head must be at least equal to the greatest dimension from the outermost bearing edge of the anchor head to the side of the anchor shank (B.4.5.2). These conditions are ordinarily satisfied easily with standard anchors.

Once the necessary steel area has been computed, sufficient embedment must be provided so that the concrete cone pullout resistance of the concrete, reduced by a capacity reduction factor, will exceed the tensile capacity of the anchor, unmodified by capacity reduction factors. The design pullout strength of the concrete is based on a maximum tensile strength of $4\sqrt{f'_c}$ (psi units), acting on the projected area of a 45° stress cone radiating toward the attachment from the bearing edge of the anchor (B.5.1.1):

$$\phi 4 \pi \sqrt{f'_c}\, h_e\, (h_e + d_h)$$

where ϕ = 0.65 (capacity reduction factor)

h_e = embedment length, measured from free surface to top of anchor head

d_h = diameter of anchor head (taken as the anchor diameter)

and other terms are as defined above.

If the axis of the anchor is close to a free edge, it is necessary to consider the consequent reduction in the projected area of the hypothetical concrete failure cone in computing the concrete cone pullout resistance. If the projected area of the cone intersects a free edge, the angle in the projected area subtended by the free edge is calculated as

$$\beta = Cos^{-1}\left[2\left(\frac{d_e}{R}\right) - 1\right]$$

where

$$R = \left(h_e + \frac{d_h}{2}\right)$$

Then the reduction in projected area is given by

$$A_L = R\left[\beta\left(\frac{R}{2}\right) - d_e \sin\left(\frac{\beta}{2}\right)\right]$$

and the effective projected area is given by

$$A_E = \pi h_e(h_e + d_h) - A_L$$

The concrete cone resistance is then that net projected area, multiplied by ($4\sqrt{f'_c}$).

To prevent bursting failure, the minimum edge distance is specified as

$$d_e \geq \frac{D}{\sqrt{\dfrac{f_{ut}}{73\sqrt{f'_c}}}}$$

where D = nominal diameter of anchor, inches

6.3 LRFD Evaluation of Anchor Design Provisions of ACI 349-85, Appendix B

The application of LRFD to reinforced concrete is discussed in Ref. 16. The underlying principles, applied to the anchor design provisions of ACI 349-85, Appendix B, are as follows:

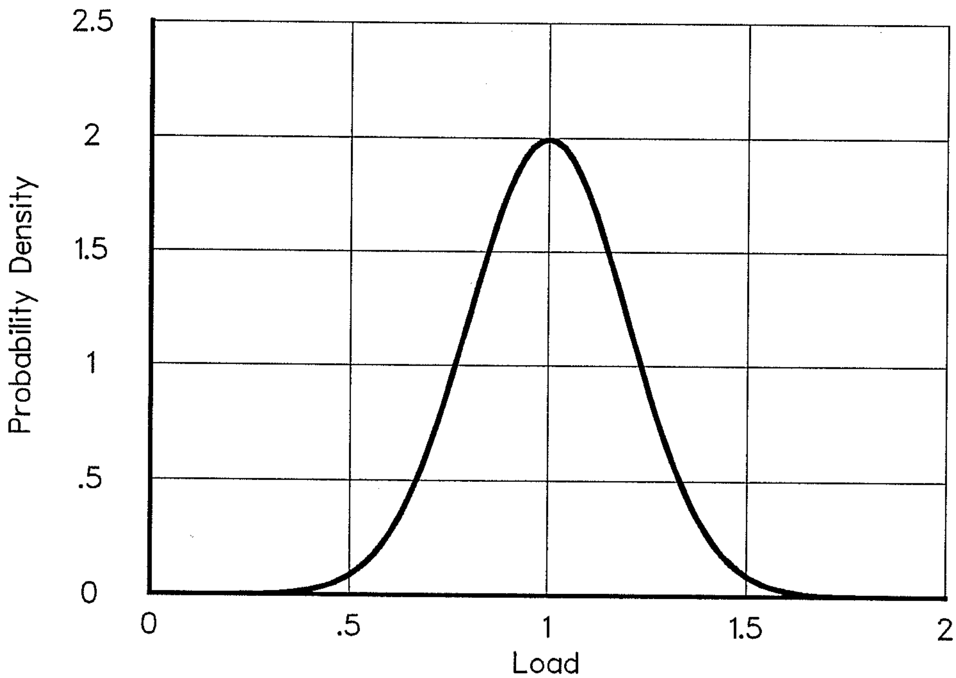

Figure 12: Hypothetical Statistical distribution of Loads

1) Loads are assumed to be statistically distributed. A hypothetical example of this distribution is presented in Fig. 12, using a normal (Gaussian)

distribution with an arbitrary mean of 1.0. The vertical axis is expressed in terms of probability density the area under the curve is 1.0. No units are specified, because the distribution is independent of the units, and because its application is dependent only on the relationship of this curve to other curves. Provided that units are consistent, they are otherwise unimportant.

2) Resistances of structural elements (in this case, a tensile anchor) are also assumed to be statistically distributed. The statistical distribution of anchor resistances depends on the statistical distributions of the ultimate tensile strength of the steel used, of the tensile strength of the concrete, of the actual embedment depth compared to that specified, of the actual edge distance compared to that specified, and so forth. For illustrative purpose only, a hypothetical example of this distribution is presented in Fig. 13. Later in this chapter, specified numerical value associated with the distribution will be derived.

3) Because loads and resistances are statistically distributed, a finite probability of failure always exists. In terms of Figs. 12 and 13, the probability of failure is given by that part of the statistical distribution of (resistance minus load) lying to the left of the vertical axis--that is, that percentage of cases for which resistance is less than load. The probability of failure can be

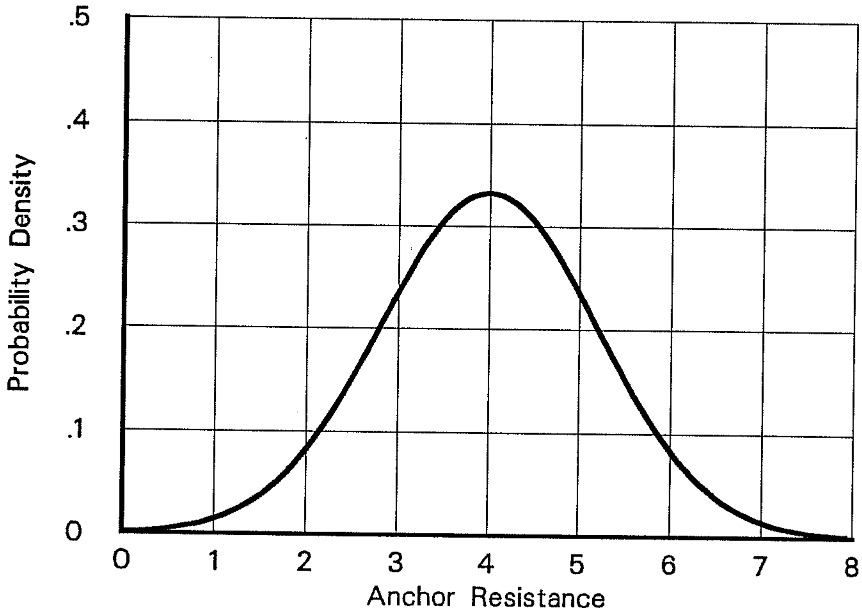

Figure 13: Hypothetical Statistical Distribution of Anchor Resistances

made very small, but cannot be eliminated. The probability of failure is sometimes described in terms of a "safety index," equal to the number of standard deviations between the mean of the resistance minus load curve, and the origin (Fig. 14).

4) A tensile anchor is designed so that its reference strength, reduced by an understrength (resistance) factor, is greater than or equal to the applied loads, increased by a load factor. The effect of these load and resistance factors is to decrease the probability of failure.

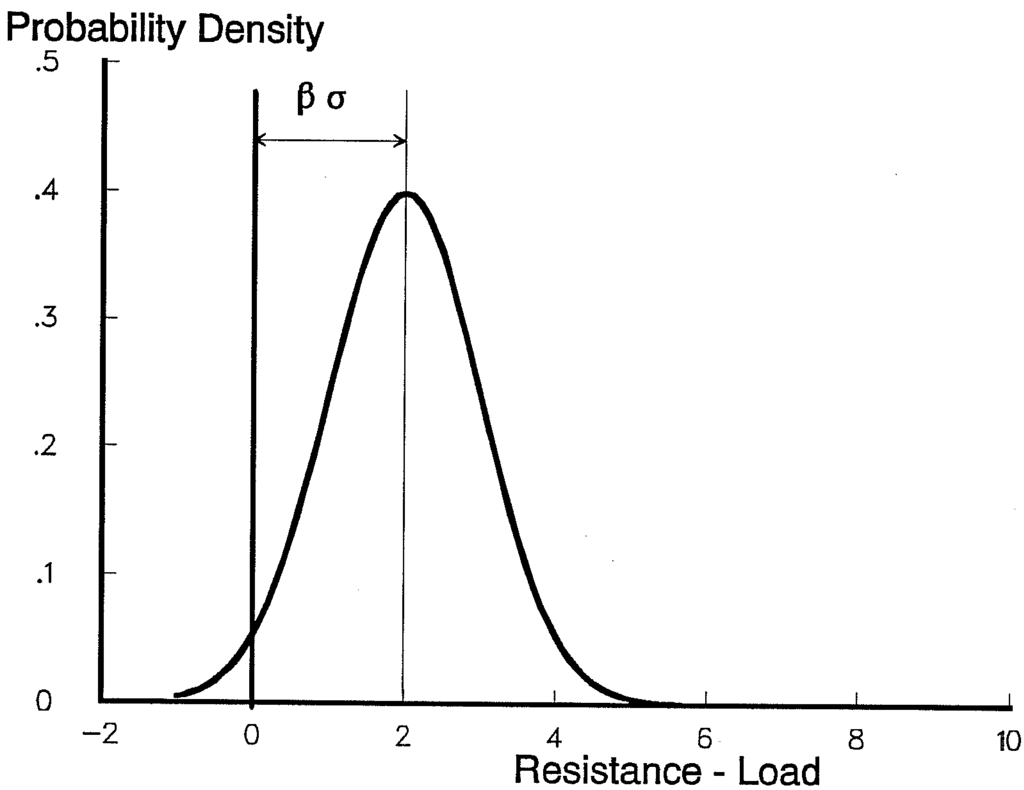

Figure 14: Hypothetical Statistical Distribution of Resistance minus Loads

5) Regardless of whether the design code describes the loads in terms of means or upper fractiles, or describes the resistances in terms of mean values, expected values, or lower fractiles, if the load and resistance factors are derived on a consistent basis, and are correlated with the actual statistical distributions of loads and resistances, the load and resistance factors can be set to achieve a desired value of the safety index (desirably low probability of failure).

6.3.1 Estimate the Statistical Distribution of Anchor Loads:

Measurements of live load on typical office buildings have shown that the statistical distribution of loads is approximately log-normal. Building codes generally specify live loads at the 95-percentile value [16]. That is, the prescribed value is greater than or equal to 95 percent of the observed load values.

No data are available regarding the statistical distribution of loads on anchors. For simplicity, it was decided to use a normal distribution, with a mean of 1.0 and a coefficient of variation of 20% (Fig. 15). The design load was taken at the 95-percentile value of the assumed average load. The 95-percentile value corresponds to 1.924 standard deviations above the mean, and hence the load is equal to 1.385. The statistical distribution of anchor loads, assumed here to follow a normal (Gaussian) distribution, could also have been computed according to other distribution, such as log-normal or extreme value. After study of the effects of different assumed distributions, it was decided to continue with the assumption of a normal distribution. This point is discussed further in Appendix K.

If the anchor loads had come from some other source, such as self-induced loads from restrained thermal shrinkage, their mean and dispersion would have been different, but the same principles could have been followed.

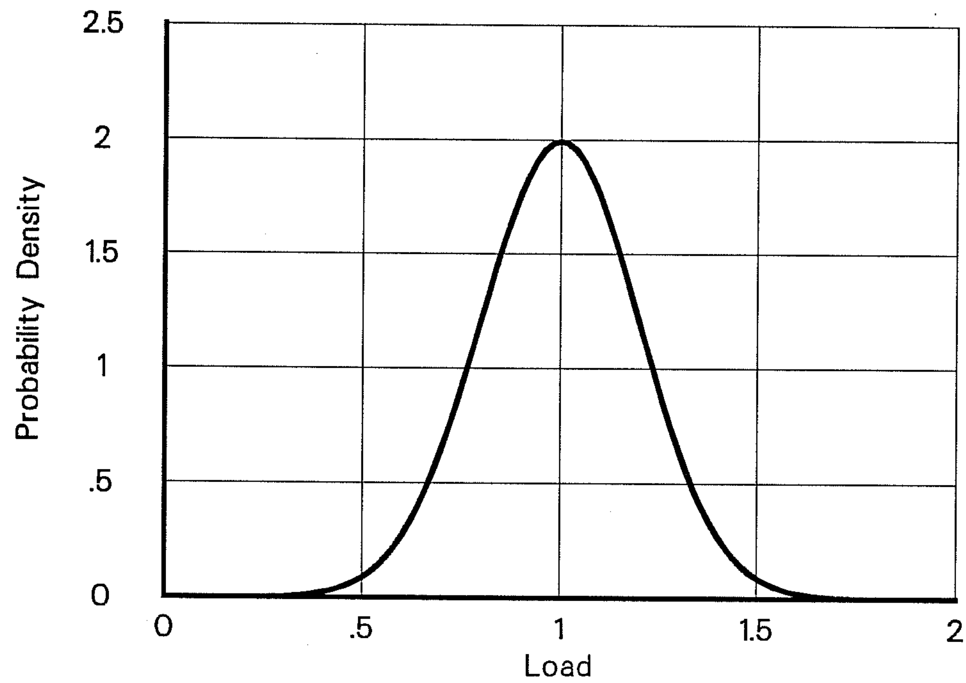

Figure 15: Assumed Statistical Distribution of Anchor Loads

6.3.2 Estimate the Statistical Distribution of Anchor Resistances as Governed by Steel:

According to <u>ACI 349-85</u> Appendix B, the required steel resistance must be greater than or equal to the factored load. The required steel resistance must be based on the greater of (A_s f_y) or (0.8 A_s f_{ut}). Generally speaking, the former will govern, as shown by

Eqs. 1 and 2. For A193-B7 steel, the ratio of f_{ut} to f_y is 1.2; that ratio was used here.

Design steel resistance based on $0.8f_{ut}$:

$$Design\ Steel\ Area = \frac{Design\ Load \times Load\ Factor}{0.8 f_{ut}}$$

$$\frac{(1.385 \times 1.7)}{0.8} = 2.943 \qquad \textbf{(1)}$$

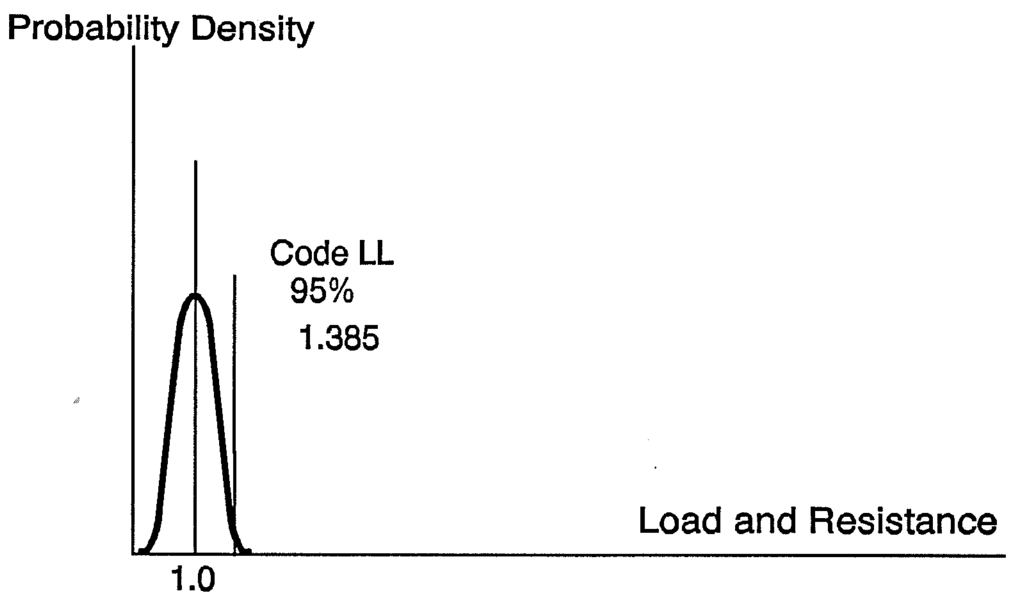

Figure 16: Calculated Distribution of Loads

Design steel resistance based on ϕf_y:

$$\text{Design Steel Area} = \frac{\text{Design Load} \times \text{Load Factor}}{0.9 \times f_{ut}} \times \frac{f_{ut}}{f_y}$$

$$\left[\frac{(1.385) \times (1.70)}{0.90}\right] \times (1.20) \;=\; 3.138 \qquad \textbf{(2)}$$

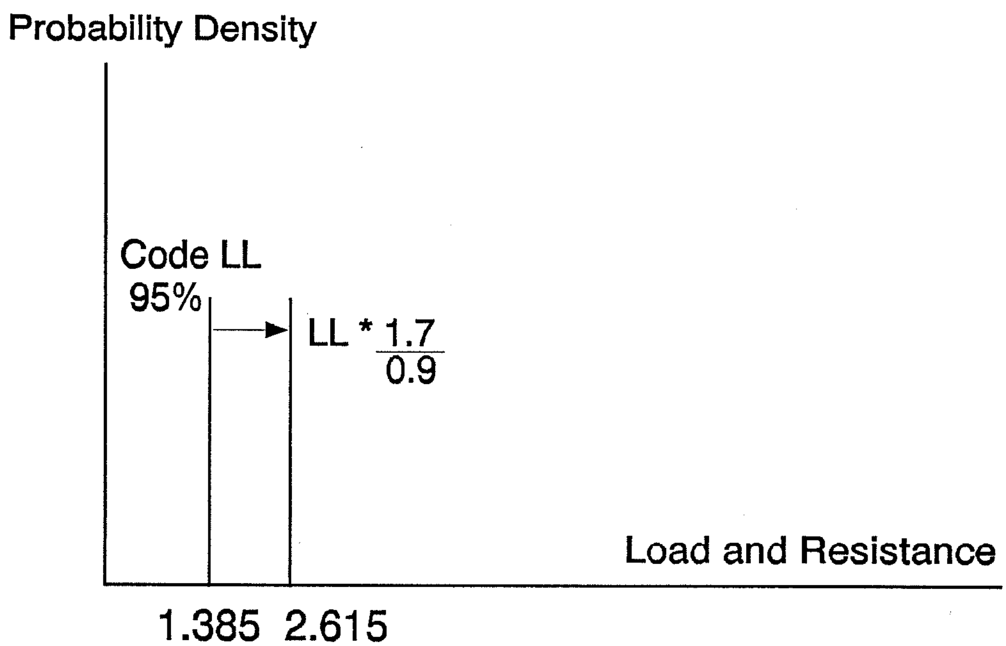

Figure 17: Required Nominal Steel Resistance

The required steel resistance is equal to the design load (the 95-percentile value of the load distribution), multiplied by the load factor for live load (1.7), and divided by ACI 349-85's ϕ factor for steel (0.90). Using the above arbitrary value of 1.0 for the average value of the load, the 95-percentile value is 1.385, as shown by Fig. 16. The resulting required nominal steel resistance is given by Eq. 3 and shown in Fig. 17.

$$\textit{Nominal Steel Resistance} = \frac{\textit{Design Load} \times \textit{Load Factor}}{\phi \textit{ factor}}$$

$$A_s f_y = \frac{(1.385) \times (1.70)}{0.90} = 2.615 \qquad \textbf{(3)}$$

If the original steel design is governed by f_y, then the theoretical steel resistance will be equal to the required nominal steel resistance (Eq. 3 and Fig. 17) multiplied by the ratio of f_{ut} to f_y. For A193-B7 steel this ratio is 1.2; that value was used here. The corresponding theoretical steel resistance is given by Eq. 1 and shown in Fig. 18. However, if the original design is governed by f_{ut}, then the theoretical steel resistance is equal to the 95-percentile load, multiplied by the load factor for live load (1.7), divided by 0.8 (the constant in the ACI 349-85 design equation $0.8A_sf_{ut}$). Eq. 2 and Fig. 18 give the theoretical steel resistance as governed by f_{ut}. Fig. 18 shows that the theoretical steel resistance based on f_{ut} (2.943) is less than that based on f_y (3.138). The latter therefore governs.

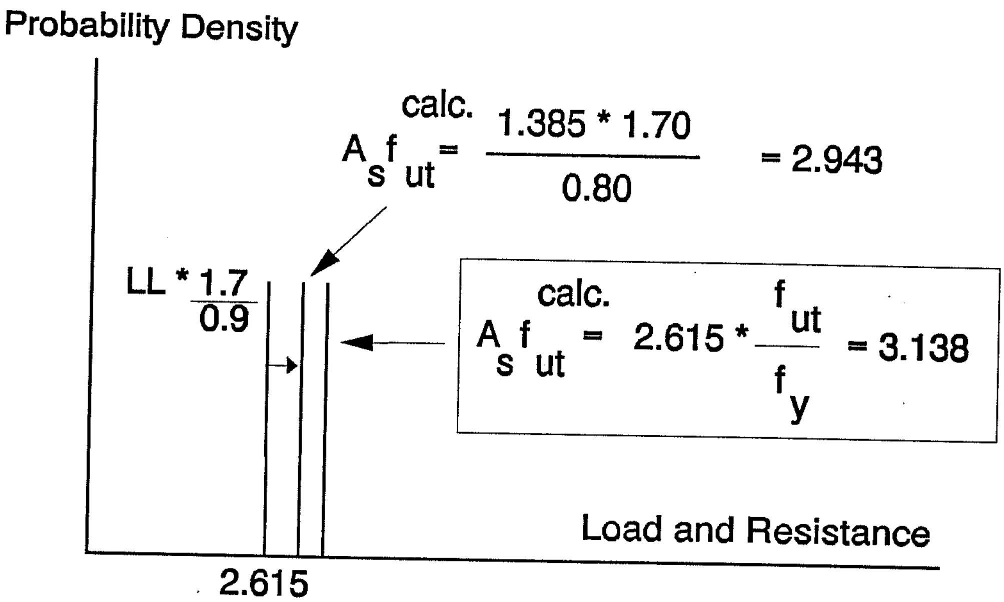

Figure 18: Theoretical Steel Resistance

The actual resistance of the steel will differ from the theoretical steel resistance, due to several factors. Chief among these is the inaccuracy of the equation used by ACI 349-85 to predict steel capacity. Using the steel failure data of Klingner and Mendonca [8] and Collins [9], the statistical distribution of actual resistances divided by the ACI 349-85 predictions had an average of 1.444, and a coefficient of variation of 0.156 for A490 and A193-B7 (high strength steel anchors). Those results are summarized in Appendix B of this thesis. They could easily be extended to data bases involving more tests, or different kinds of anchors. Based on the results used here and using the governing equation, the resulting average actual steel resistance is given by Eq. 4 and shown by Fig. 19.

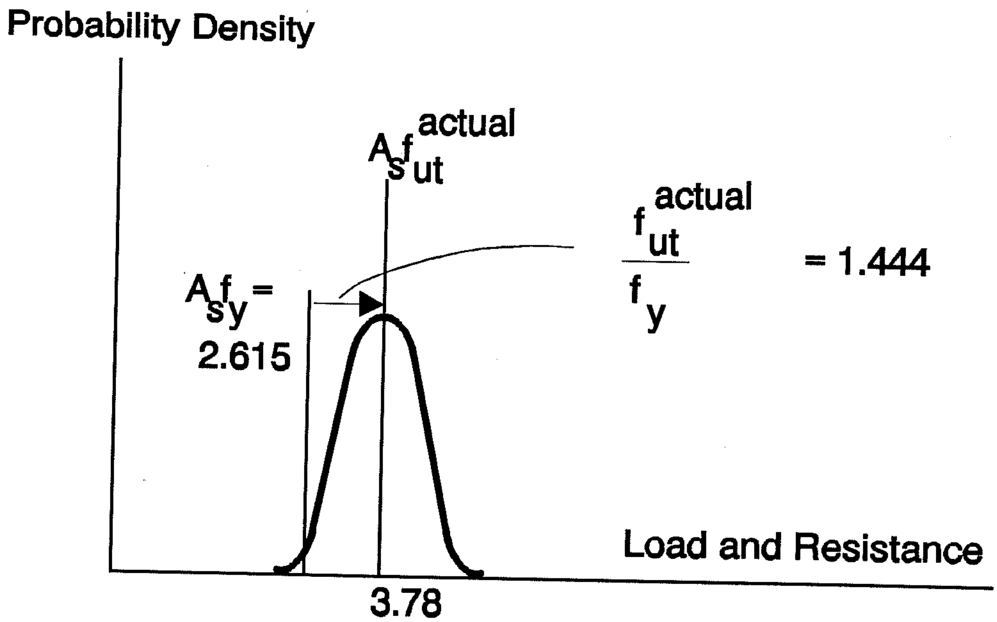

Figure 19: Actual Steel Resistance

$$\textit{Actual Steel Resistance} = \textit{Nominal Steel Resistance} \times \frac{\textit{Actual mean}}{\textit{Nominal mean}}$$

$$= \left[\frac{(1.385) \times (1.70)}{0.90}\right] \times (1.444) = 3.78 \qquad \textbf{(4)}$$

The corresponding coefficient of variation, 0.156, does not change. The resulting statistical distribution of actual steel resistances consistent with the arbitrary load is therefore as shown in Fig. 20.

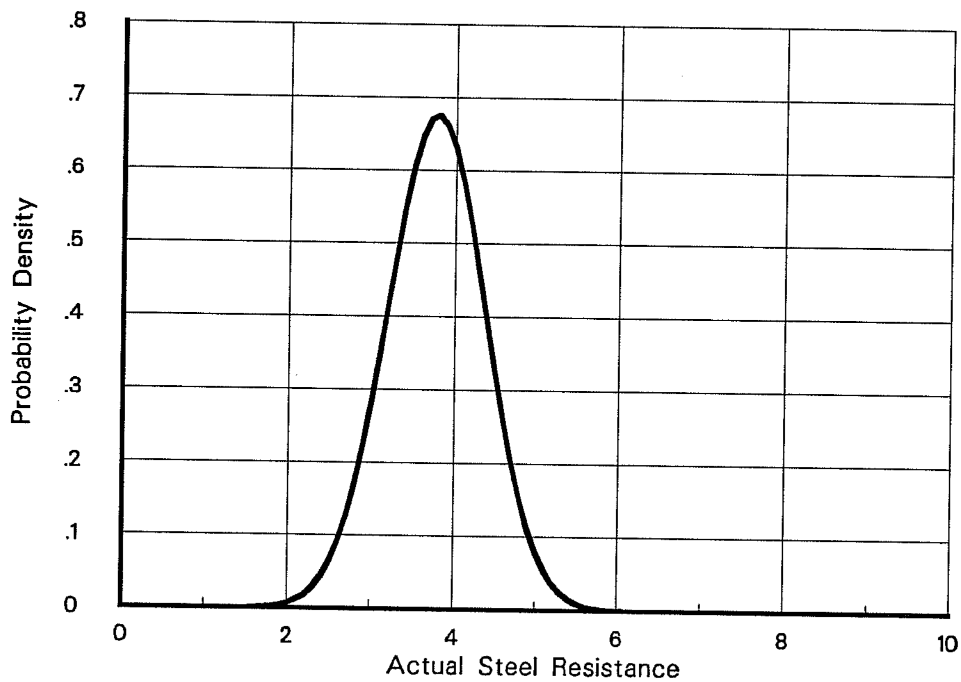

Figure 20: Statistical Distribution of Actual Anchor Resistance as Governed by Steel (Mean = 3.78)

6.3.3 Estimate the Statistical Distribution of Anchor Resistances as Governed by Concrete:

Because the ACI 349-85 Appendix B requires sufficient embedment to develop the steel capacity, the nominal capacity of the concrete depends on the steel capacity, not the design load. ACI 349-85 Appendix B requires that:

$$A_s f_{ut} \leq \phi \times (Concrete\ Capacity)$$

Using the same arbitrary average load value from Fig. 16, the average nominal yield resistance of the anchor steel is given by Eq. 5. According to the provisions of ACI 349-85, the required nominal strength of the anchor, as governed by concrete (theoretical concrete resistance), and reduced by an understrength factor of 0.65, must at least equal the specified ultimate tensile capacity of the anchor steel. In this thesis, the required nominal yield capacity of the anchor steel (yield strength of the anchor steel times the tensile stress area) has been computed above as 2.615 (based on an assumed mean load of 1.0). The required nominal capacity of the anchor will be 2.615, multiplied by the ratio of specified ultimate strength to specified yield strength.

$$\frac{(1.385)\times(1.70)}{0.90} = 2.615 \qquad \textbf{(5)}$$

Most anchor steels have a ratio of specified ultimate strength to specified yield strength of about 1.2. In particular, for A193-B7 steel, f_{ut} is 125 ksi and f_y is 105 ksi, giving a ratio of 1.20. That value is used in this thesis. If desired, each anchor's actual variation of specified yield to specified ultimate strength could be taken into account.

The required nominal strength of the anchor as governed by concrete will therefore be as given by Eq. 6 and shown in Fig. 21:

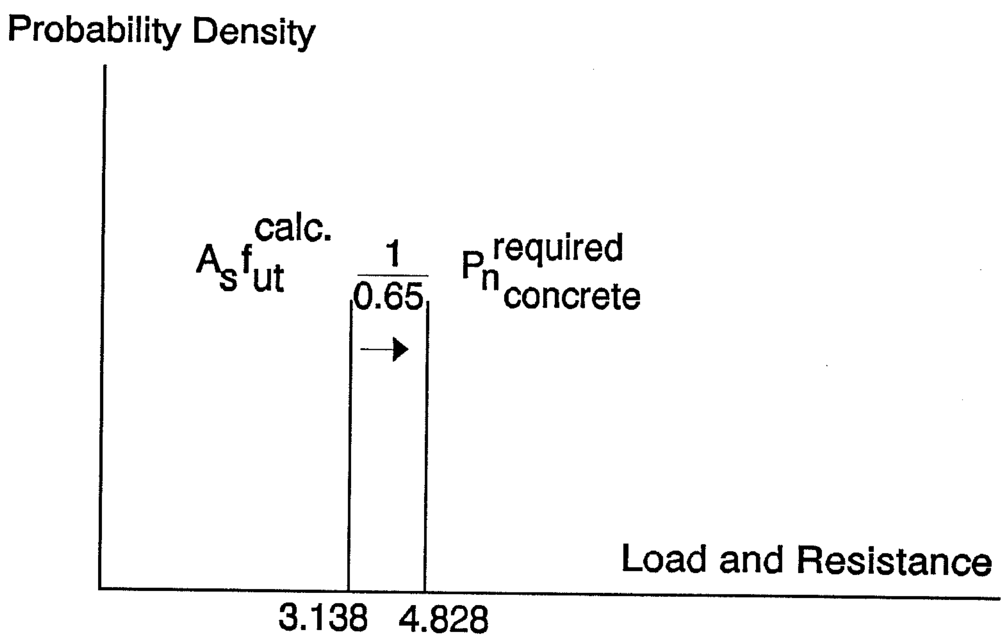

Figure 21: Theoretical Concrete Resistance

$$2.615 \times 1.2 \times \frac{1}{0.65} = 4.828 \quad \textbf{(6)}$$

The actual resistance of the anchor as governed by concrete will differ from the required nominal strength, for several reasons. Chief among these is the accuracy of the equation used by ACI 349-85 to predict concrete capacity. Because the accuracy of this equation varies with embedment depth, the ratio between probable and nominal concrete capacity also varies with embedment depth, and must be computed over different ranges of embedment depths.

The following ranges of embedment depths are considered: 0.01-1 in. (0.3-25.4 mm); 1.01-2 in. (25.5.7-50.8 mm); 2.01-3 in. (51.1-76.2 mm); 3.01-4 in. (76.5-101.6 mm); 4.01-5 in (101.9-127.0 mm); 5.01-6 in. (127.3-152.4 mm); 6.01-8 in. (152.7-203.2 mm); and greater than 8.01 in. (203.5 mm). Although the exact procedure would have been to determine the probability of failure for each discrete embedment depth, this was not possible due to variability and/or lack of data for certain embedment depths. In particular, some of the deeper embedments had only a few tests.

Based on the 801 test results for concrete failure studied here, the mean and coefficient of variation for actual concrete resistance divided by predicted concrete resistance (ACI 349-85, Appendix B) are listed in Table 3 below for each range of embedment depth. Note that these values are independent of whether U.S. or SI units are used.

For example, for embedment depths from 5.01 to 6.0 inches, the distribution of actual concrete capacities divided by the ACI 349-85 predictions had a mean value of

1.10 and a coefficient of variation of 0.159. The corresponding actual concrete strength distribution will therefore have a mean value as given by Eq. 7 and shown in Fig. 22.

$$4.828 \times 1.10 = 5.310 \quad \textbf{(7)}$$

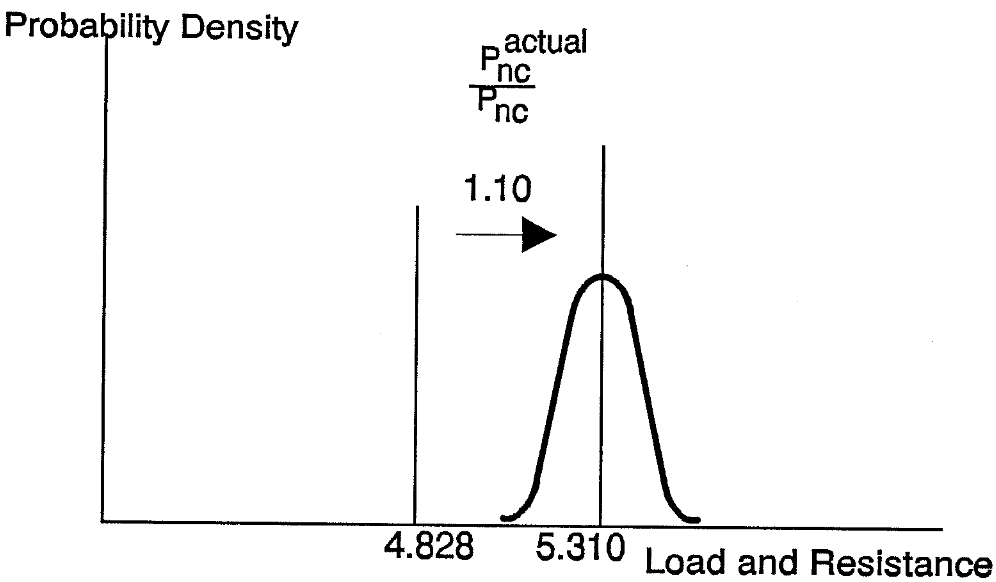

Figure 22: Actual Concrete Resistance, Embedment Depths 5.01-6.0 in

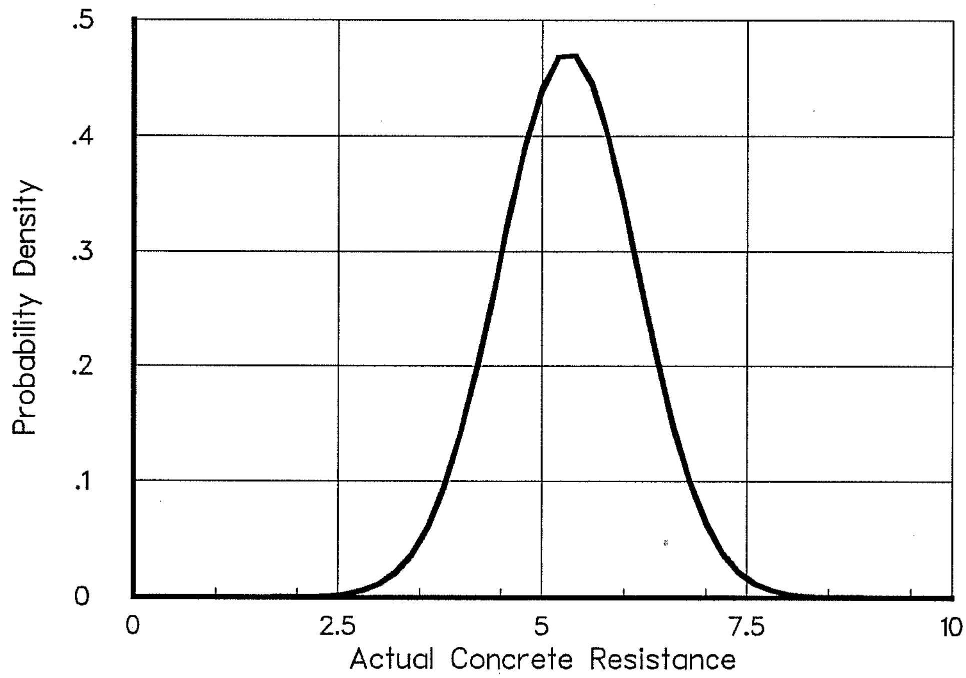

Figure 23: Statistical Distribution of Actual Anchor Resistance as Governed by Concrete ACI 349-85 (Embedment Depths of 5.01-6.0 in)

The coefficient of variation will remain 0.159. The resulting statistical distribution of actual concrete resistances, consistent with the arbitrary mean load of 1.0 is therefore as shown in Fig. 23.

6.3.4 Use Monte Carlo Technique to Compute Statistical Distribution of (Resistance minus Load):

The load and resistance distributions of Figs. 16 and 20, plus the resistance values of Table 3, are now

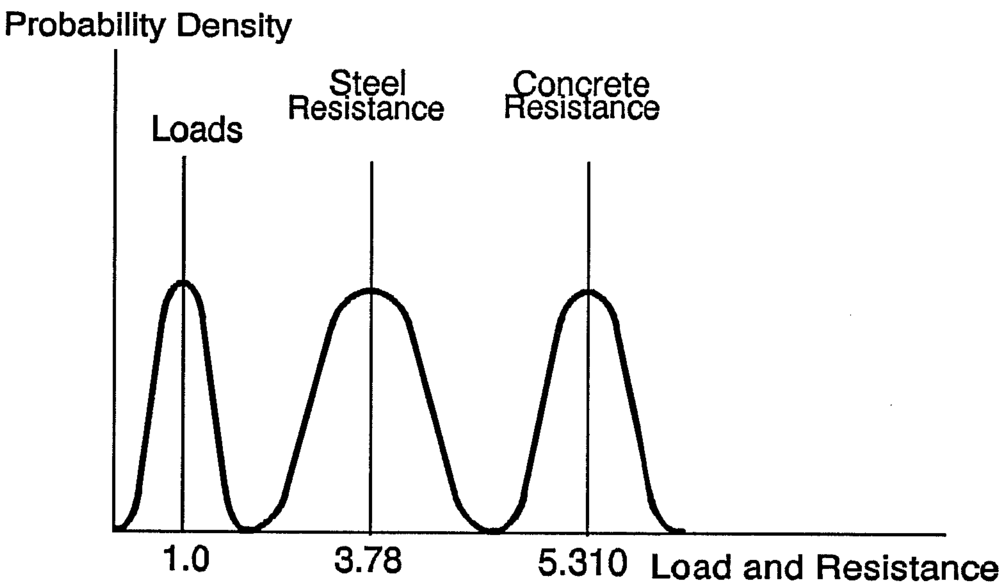

Figure 24: Loads, Steel Resistance, and Concrete Resistance Distribution (Embedment Depths 5.01-6.0 in)

used to compute the statistical distribution of (Resistance minus Load) and the probability of failure for a range of embedment depths, using the Monte Carlo technique. For example for embedment depths between 5.01-6.0 inches, The loads, steel resistance, and concrete resistance distributions are shown in Fig. 24.

The numerical calculations, carried out by computer as described in Appendix G [17], are outlined below:

1) For each embedment depth, the statistical distribution is discretized into a histogram. Each discrete load or resistance value is assigned an integer number of cells, in proportion to the probability of occurrence of that load or resistance value. In this case, the total number of cells is selected as 10,000. A total of 201 discrete values of load or resistance are selected, and the integer number of cells corresponding to each value is selected so that the total histogram contains 10,000 cells. In other words, each histogram is represented by different integer numbers of 201 values, each of which occurs in the set of 10,000, in proportion to its occurrence in the original statistical distribution. The result is three sets of values: one for loads; one for probable steel resistances; and one for probable concrete resistances.

2) Three sets of 10,000 random integers between 1 and 10,000 are created, using a pseudo-random number generation utility with three different seeds. Each combination of random integers, one from each set, represents a combination of load, steel resistance, and concrete resistance. The combinations are selected as follows: The i^{th} value of each set of random numbers is selected. Suppose, for example, that those three values are 203, 999, and 5. Therefore, the 203^{rd} value of the load is combined with the 999^{th} value of the steel resistance and the 5^{th} value of the concrete resistance. Because each

load and resistance value occurs in proportion to its relative probability, the combinations so obtained represent real combinations of load and

Table 3: Mean and Coefficient of Variation of Actual Capacity Divided by Predicted Concrete Capacity for Each Method, as a Function of Embedment Depth

Embedment Depth	ACI 349-85		Variable Cone		Kappa	
	Mean	COV	Mean	COV	Mean	COV
0.01-1 in 0.3-25.4 mm	2.24	0.225	0.904	0.211	1.00	0.198
1.01-2 in 25.7-50.8 mm	1.78	0.205	0.882	0.197	0.977	0.204
2.01-3 in 51.1-76.2 mm	1.41	0.203	0.813	0.203	0.965	0.208
3.01-4 in 76.5-101.6 mm	1.29	0.155	0.924	0.160	0.992	0.156
4.01-5 in 101.9-127.0 mm	1.15	0.166	1.08	0.179	1.03	0.185
5.01-6 in 127.3-152.4 mm	1.10	0.159	1.10	0.159	1.05	0.157
6.01-8 in 152.7-203.2 mm	1.05	0.156	1.05	0.156	1.04	0.170
> 8.01 in > 203.2 mm	0.885	0.206	0.885	0.206	0.993	0.179

resistance. The minimum value of resistance (lesser of steel and concrete), minus the value of load, represents a real (Resistance minus Load) value.

3) This process is repeated for each of the 10,000 sets of three random integers, and 10,000 (Resistance minus Load) values are obtained. These values are checked to ensure that the means and standard deviations of the discretized histograms match the target values.

4) The (Resistance minus Load) values so obtained are also assumed to be normally distributed. Their mean and standard deviation are calculated, as is the corresponding probability of failure (the area of the probability distribution lying to the left of the vertical axis), as shown in Fig. 25.

5) Each time the computer program of Appendix G is run, slightly different values of the safety index and the corresponding probability of failure are obtained, because different combinations of random numbers are selected. However, the values do not differ significantly from run to run. The results of the Monte Carlo analyses are shown in Table 4 for both U.S. and SI units, and in Figs. 28 and 29 for U.S. and SI units, respectively. Note that the probability of failure is independent of units. In Figs. 28, 29 and similar figures below, the embedment lengths are given approximately due to lack of space. For example, the embedment range

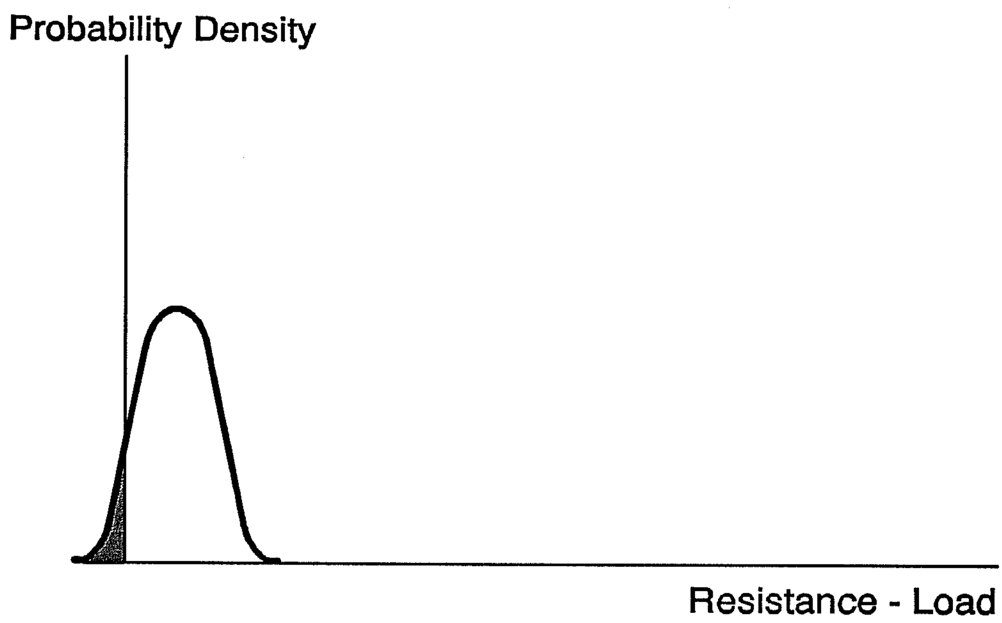

Figure 25: Probability of Failure under Known Loads

0.01 - 1 inch (0.3 - 24.5 mm) is indicated approximately as 0 - 1 inch (0-25 mm). The actual ranges are those shown in Tables 2 through 5.

Using the ACI 349-85 theory, the probability of failure under known loads is calculated based on the mean and standard deviation for each range of embedment depths. Table 4 and Figs. 28 and 29 show that the probability of failure under known loads is of the order of 10^{-6} for embedment depths up to 8 inches (203 mm), and of the order of 10^{-5} for embedment depth larger than 8 inches (203 mm).

6.3.5 Use Monte Carlo Analysis to Compute Probability of Concrete Failure under Unlimited Load:

A Monte Carlo analysis similar to that conducted above can be used to predict the probability of concrete failure under unlimited load, of a single tensile anchor designed by ACI 349-85. This is significant because the probability of failure of a ductile multiple-anchor attachment is related to the probability that all anchors will develop their tensile capacity as governed by steel, without experiencing concrete failure.

The resistance distribution of Fig. 13 and the resistance values of Table 3 are now used to compute the statistical distribution of (concrete resistance minus steel resistance) and the corresponding probability of concrete failure under unlimited loads for a range of embedment depths, using the Monte Carlo technique (Fig. 27).

1) The discrete histogram for actual steel resistance is generated based on actual steel resistance, and is like that of Fig. 20. The discrete histogram for concrete, also based on actual concrete resistance, does not change from that of Fig. 23.

2) Only two sets of 10,000 random numbers need be generated to compute the histogram of (Concrete Resistance minus Steel Resistance) (Fig. 26).

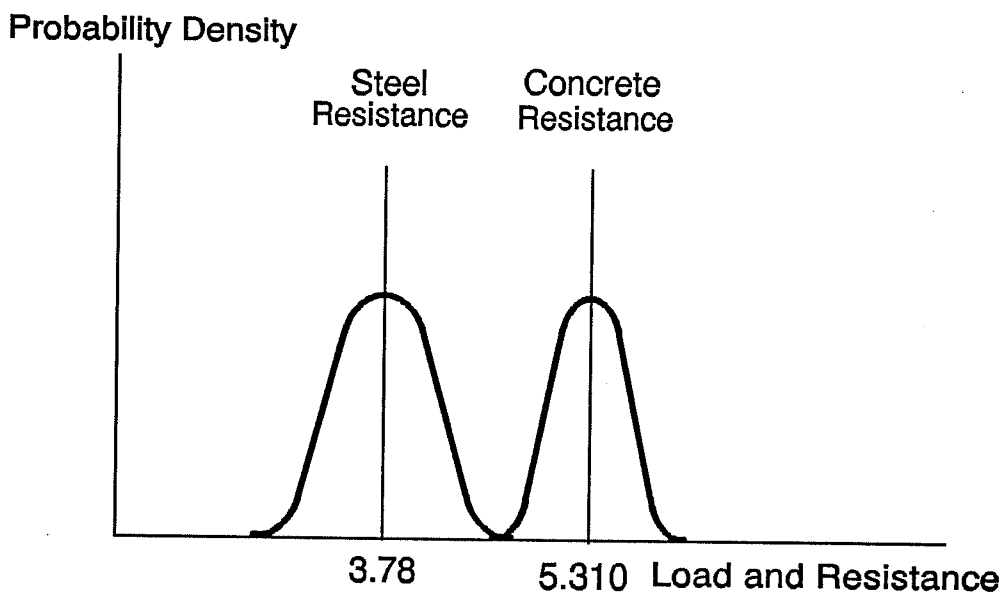

Figure 26: Concrete Resistance and Steel Resistance (Embedment Depths 5.01-6.0 in)

3) The (Concrete Resistance minus Steel Resistance) values so obtained are also assumed to be normally distributed. Their mean and standard deviation are calculated, as is the corresponding probability of failure (the area of the probability distribution lying to the left of the vertical axis) as shown in Fig. 27.

The numerical calculations are carried out by computer as described in Appendix H. The results of the Monte Carlo analysis are shown in Table 5 for both U.S.

and SI units, and in Figs. 30 and 31 for U.S. and SI units, respectively.

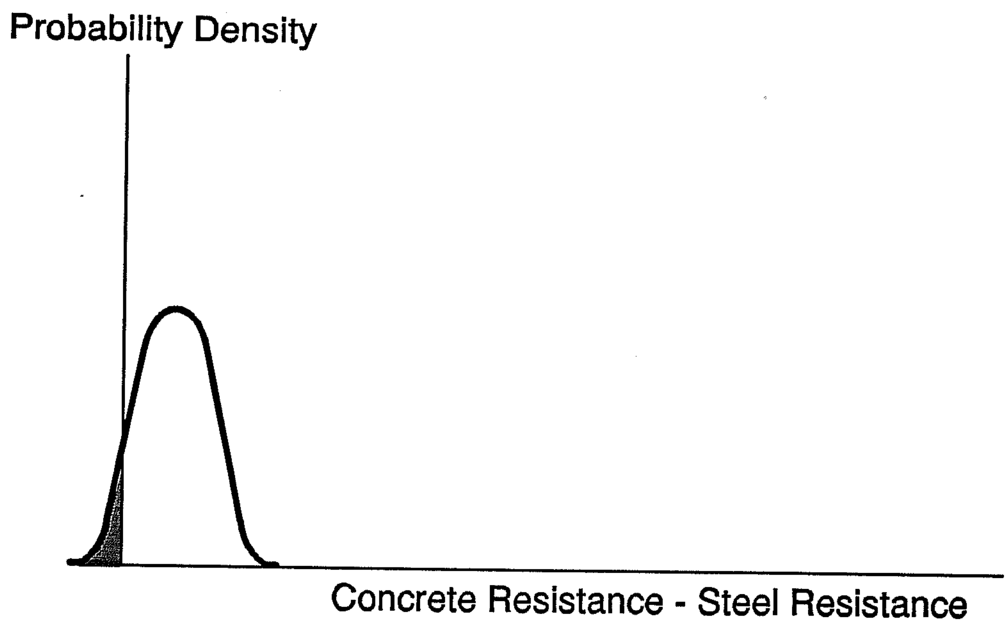

Figure 27: Probability of Concrete Failure under unlimited Loads

6.4 LRFD Evaluation of Anchor Design Provisions of Variable-Angle Cone Theory:

The underlying principles of Load and Resistance Factor Design, applied to the variable-angle cone theory discussed above, are identical to those discussed in Section 6.3 above. The only difference lies in the computation of the probable concrete resistance associated with the required nominal capacity as governed by concrete failure. Therefore, only that section will be discussed.

6.4.1 Estimate the Statistical Distribution of Anchor Resistances as Governed by Concrete:

Using the arbitrary average load value, the average nominal yield resistance of the anchor steel is given by Eq. 5 above. The required nominal strength of the anchor as governed by concrete will therefore be as given by Eq. 6 above.

The actual resistance of the anchor as governed by concrete will differ from the required nominal strength, for the same reasons discussed above. Based on this study of 801 tensile anchor tests failing in concrete, the statistical distribution of actual resistances divided by the predictions of the variable-angle cone theory had mean values and coefficients of variation as given in Table 3, for various ranges of embedment depths.

6.4.2 Use Monte Carlo Technique to Compute Statistical Distribution of (Resistance minus Load):

For each range of embedment depths, the load and resistance distributions discussed above are now used to compute the statistical distribution of (Resistance minus Load), using the Monte Carlo technique. The numerical calculations are carried out as described in Section 6.3. The results are shown in Table 4 for both U.S. and SI units, and in Figs. 28 and 29 for U.S. and SI units, respectively.

6.4.3 Use Monte Carlo Analysis to Compute Probability of Concrete Failure under Unlimited Load:

For each range of embedment depths, the steel and concrete resistance distributions discussed above are now used to compute the probability of concrete failure under unlimited load, of a single tensile anchor designed by the variable-angle cone theory. The numerical calculations are carried out by computer as described in Appendix H. The results of the Monte Carlo analysis are shown in Table 5 for both U.S. and SI units, and in Figs. 30 and 31 for U.S. and SI units, respectively.

6.5 LRFD Evaluation of Anchor Design Provisions of Kappa Theory

The underlying principles of Load and Resistance Factor Design, applied to the Kappa theory discussed above, are identical to those discussed in Section 6.3 above. The only difference lies in the computation of the probable concrete resistance associated with the required nominal capacity as governed by concrete failure. Therefore, only that section will be discussed.

6.5.1 Estimate the Statistical Distribution of Anchor Resistances as Governed by Concrete:

Using the arbitrary average load value, the average nominal yield resistance of the anchor steel is given by Eq. 5 above. The required nominal strength of the anchor as governed by concrete will therefore be as given by Eq. 6 above.

The actual resistance of the anchor as governed by concrete will differ from the required nominal strength, for the same reasons discussed above. Based on this study of 801 tensile anchor tests failing in concrete, the statistical distribution of actual resistances divided by the predictions of the Kappa theory had mean values and coefficients of variation as given in Table 3, for various ranges of embedment depths.

6.5.2 Use Monte Carlo Technique to Compute Statistical Distribution of (Resistance minus Load):

For each range of embedment depths, the load and resistance distributions discussed above are now used to compute the statistical distribution of (Resistance minus Load), using the Monte Carlo technique. The numerical calculations are carried out as described in Section 6.3. The numerical calculations are carried out as described in Section 6.3. The results are shown in Table 2 for both U.S. and SI units, and in Figs. 28 and 29 for U.S. and SI units, respectively.

6.5.3 Use Monte Carlo Analysis to Compute Probability of Concrete Failure under Unlimited Load:

For each range of embedment depths, the steel and concrete resistance distributions discussed above are now used to compute the probability of concrete failure under unlimited load, of a single tensile anchor designed by the Kappa theory. The numerical calculations are carried out by computer as described in Appendix H. The results of the Monte Carlo analysis are shown in Table 5 for both U.S. and SI units, and in Figs. 30 and 31 for U.S. and SI units, respectively.

6.6 General Limitations of These Analyses

These analyses have the following limitations:

1) Loads and are assumed to be normally distributed. For known loads, safety factors computed by the analysis could increase or decrease as a result of different assumed distributions. For unlimited loads, probabilities of failure computed by this analysis are independent of the assumed load distribution.

2) Based on a study of different possible distributions (Appendix K), resistances were taken as normally distributed.

3) Actual concrete strength is assumed to equal the specified value. This assumption is conservative, because actual concrete strength usually exceeds that specified.

4) A single representative value is used for the ratio of specified ultimate steel strength to specified yield strength. This could be made more accurate by computing it separately for each anchor in the data base.

5) Typical field construction variations in embedment depth and edge distance are not taken into account.

6) The specimens comprising this data base involve essentially uncracked concrete. Effects of cracking would be expected to reduce the anchor capacity as governed by concrete, and would increase the probabilities of failure calculated here.

7) The analysis is intended to apply to single tensile anchors only. Safety analysis of multiple-anchor attachments can be conducted by extending the principles followed here. Care should be taken to correctly account for the effect of overlapping stress cones on the actual strength as governed by concrete, and for the effect of failure of a single anchor on the behavior of the entire connection.

6.7 Discussion of LRFD Results

6.7.1 Probability of Failure under Known Loads:

Under unlimited loads, the probability of failure associated with each theory was evaluated. The results are given in Table 4, and are shown in the form of bar charts in Figs. 28 and 29 in terms of U.S. and SI units respectively.

Because of the Monte Carlo technique, computed failure probabilities vary from run to run. When computed probabilities are small (on the order of 10^{-6}), the probabilities can change by ± 25% from run to run. When computed probabilities are larger (on the order of 10^{-2}), the change from run to run is less than ± 5%. The graphs show that anchors designed using the concrete capacity provisions of any of the three theories have probabilities of failure less than 2×10^{-5} at all embedment depths. For embedment depths of 8 inches or less (203.2 mm), the probabilities of failure associated with the ACI 349-85 and the Kappa theory are of the same order of magnitude. Given the level of uncertainty discussed above, neither theory is shown to be clearly superior.

However, for embedment depths greater than 8 inches (203.2 mm), the probability of failure associated with the Kappa theory is an order of magnitude lower than that of the ACI 349-85.

All three theories for computing concrete cone capacity produce final designs with probabilities of failure between 2 x 10^{-5} and 1 x 10^{-6}. These probabilities correspond to β between 4.2 and 4.7. The Kappa theory has probabilities of failure not exceeding 5 x 10^{-6}, and is the most consistent theory for all ranges of embedment depths.

Current design practice for reinforced concrete structures with average consequences of failure accepts designs giving β values of 3.0 to 3.5 [18]. All three theories are more conservative than this standard. Based on the Monte Carlo analysis, it may therefore be concluded that all three concrete capacity theories give a sufficiently low probability of failure for single anchors under known loads, when used in conjunction with current ACI 349-85 load and understrength factors.

6.7.2 Probability of Concrete Failure under Unlimited Loads:

Under unlimited loads, the probability of concrete failure associated with each theory was evaluated. Note that this probability is independent of the statistical distribution assumed for the loads. The results are given in Table 5, and are shown in the form of bar charts in Figs. 30 and 31 in terms of U.S. and SI units respectively.

Because of the Monte Carlo technique, computed failure probabilities vary from run to run. When computed probabilities are small (on the order of 10^{-6}), the probabilities can change by ± 25% from run to run. When computed probabilities are larger (on the order of 10^{-2}), the change from run to run is less than ± 5%. Those figures show that anchors designed using the concrete capacity provisions of any of the three theories have probabilities of concrete failure under unlimited loads less than 2×10^{-2} for all embedment depths. The only exception is the variable-angle cone theory at embedment depths of 2.01-3.0 inches (51.1-76.2 mm).

The probability of concrete failure under unlimited loads associated with the ACI 349-85 theory is lower than that of the Kappa theory for the following ranges of embedment depths: 0.01-3.0 inches (0.3-76.2 mm); 4.01-5.0 inches (101.9-127.0 mm); 6.01-8.0 inches (152.7-203.2 mm). For embedment depths of 3.01-4.0 inches (76.5-101.6 mm) the probabilities of concrete failure under unlimited loads are approximately equal for the two theories. However, for embedment depths of 5.01-6.0 inches (127.3-152.1 mm) and for embedment depths greater than 8.01 inches (203.2 mm), the probabilities of concrete failure under unlimited loads associated with the Kappa theory are lower than those of the ACI 349-85 theory. Note that the Kappa theory at its worst gives a probability of concrete failure under unlimited loads of 9.1×10^{-3} for embedment depths of 2.01-3.0 inches (51.1-76.2 mm); The theory of ACI 349-85 at its worst gives a probability of 4.2×10^{-2} for embedment depths greater than 8.01 inches

(203.2 mm); and the variable-angle cone theory at its worst gives a probability of failure of 0.238 for embedment depths of 2.01-3.0 inches (51.1-76.2 mm).

For all but one embedment depth range, all three theories for computing concrete cone capacity produce final designs with probabilities of concrete failure, under unlimited loads, between 5 x 10^{-2} and 1 x 10^{-11}. The exception to this is the variable-angle cone theory applied to embedment depths between 2.01 and 3.0 inches (51.1-76.2 mm), with a probability of failure of 0.238. Using the same terminology developed for (Resistance minus Load) calculations, these probabilities would correspond to β values between 1.8 and 6.5.

In assessing the significance of these probabilities of concrete failure under unlimited loads, it must be pointed out that the probability of occurrence of an unlimited load is itself very small. If anchors are designed using the procedure of <u>ACI 349-85</u> Appendix B and using the concrete capacity theory of <u>ACI 349-85</u> Appendix B, the probability of concrete failure under any possible loads is less than 5%. Using the Kappa theory, the probability of concrete failure under any possible loads is less than 1%. While these probability are not infinitesimal, they are believed acceptable for such an extreme condition.

Table 4: Results of Monte Carlo Analysis for Probability Failure under Known Loads

Embedment Depth	ACI 349-85		Variable Cone		Kappa	
	ß	Prob. of Failure	ß	Prob. of Failure	ß	Prob. of Failure
0.01-1 in 0.3-25.4 mm	4.51	3.2×10^{-6}	4.18	1.5×10^{-5}	4.51	3.3×10^{-6}
1.01-2 in 25.7-50.8 mm	4.53	3.0×10^{-6}	4.39	5.7×10^{-6}	4.43	4.7×10^{-6}
2.01-3 in 51.1-76.2 mm	4.53	3.0×10^{-6}	4.21	1.3×10^{-5}	4.42	4.9×10^{-6}
3.01-4 in 76.5-101.6 mm	4.57	2.5×10^{-6}	4.74	1.1×10^{-6}	4.66	1.6×10^{-6}
4.01-5 in 101.9-127.0 mm	4.61	2.0×10^{-6}	4.65	1.7×10^{-6}	4.62	1.9×10^{-6}
5.01-6 in 127.3-152.1 mm	4.59	2.2×10^{-6}	4.65	1.6×10^{-6}	4.67	1.5×10^{-6}
6.01-8 in 152.7-203.2 mm	4.65	1.7×10^{-6}	4.67	1.5×10^{-6}	4.64	1.8×10^{-6}
> 8.01 in > 203.2 mm	4.19	1.4×10^{-5}	4.36	6.5×10^{-6}	4.64	1.7×10^{-6}

Table 5: Results of Monte Carlo Analysis for Probability of Concrete Failure under Unlimited Loads

Embedment Depth	ACI 349-85		Variable Cone		Kappa	
	ß	Prob. of Failure	ß	Prob. of Failure	ß	Prob. of Failure
0.01-1 in 0.3-25.4 mm	3.89	5.4×10^{-5}	1.79	3.7×10^{-2}	2.93	1.7×10^{-3}
1.01-2 in 25.7-50.8 mm	4.17	1.5×10^{-5}	1.93	2.7×10^{-2}	2.57	5.1×10^{-3}
2.01-3 in 51.1-76.2 mm	3.92	4.5×10^{-5}	0.71	0.238	2.36	9.1×10^{-3}
3.01-4 in 76.5-101.6 mm	6.61	1.9×10^{-11}	5.27	7.9×10^{-8}	6.58	2.4×10^{-11}
4.01-5 in 101.9-127.0 mm	5.36	4.1×10^{-8}	4.21	1.3×10^{-5}	3.69	1.1×10^{-4}
5.01-6 in 127.3-152.1 mm	6.12	4.6×10^{-10}	6.06	6.8×10^{-10}	6.42	6.9×10^{-11}
6.01-8 in 152.7-203.2 mm	6.39	8.6×10^{-11}	6.46	5.3×10^{-11}	4.73	1.1×10^{-6}
> 8.01 in > 203.2 mm	1.72	4.2×10^{-2}	1.71	4.4×10^{-2}	3.86	5.7×10^{-5}

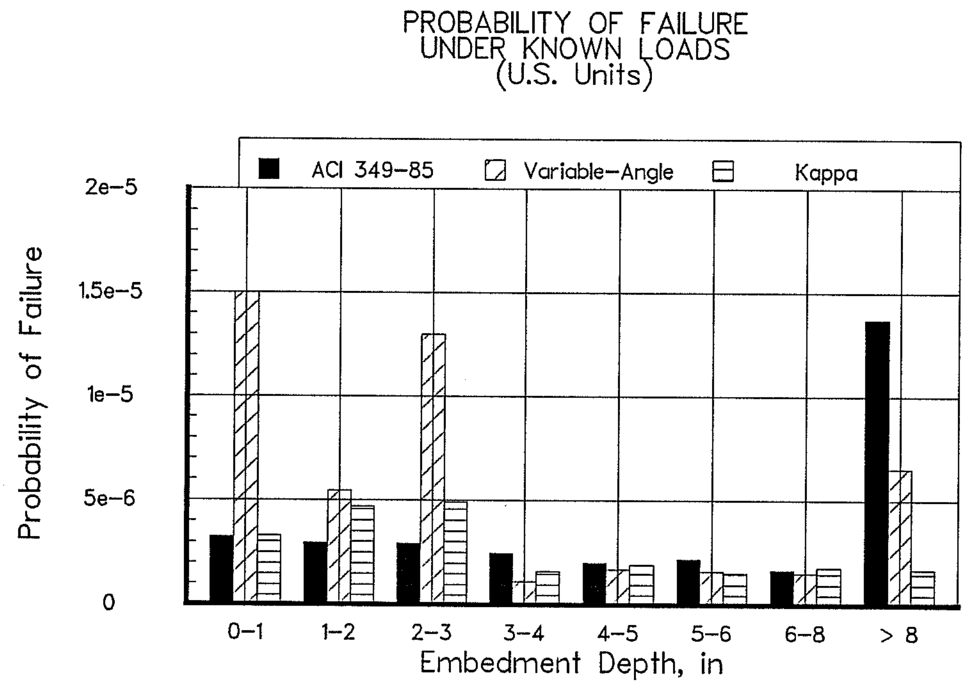

Figure 28: Comparison of Probability of Failure under Known Loads Using Each Theory (SI Units)

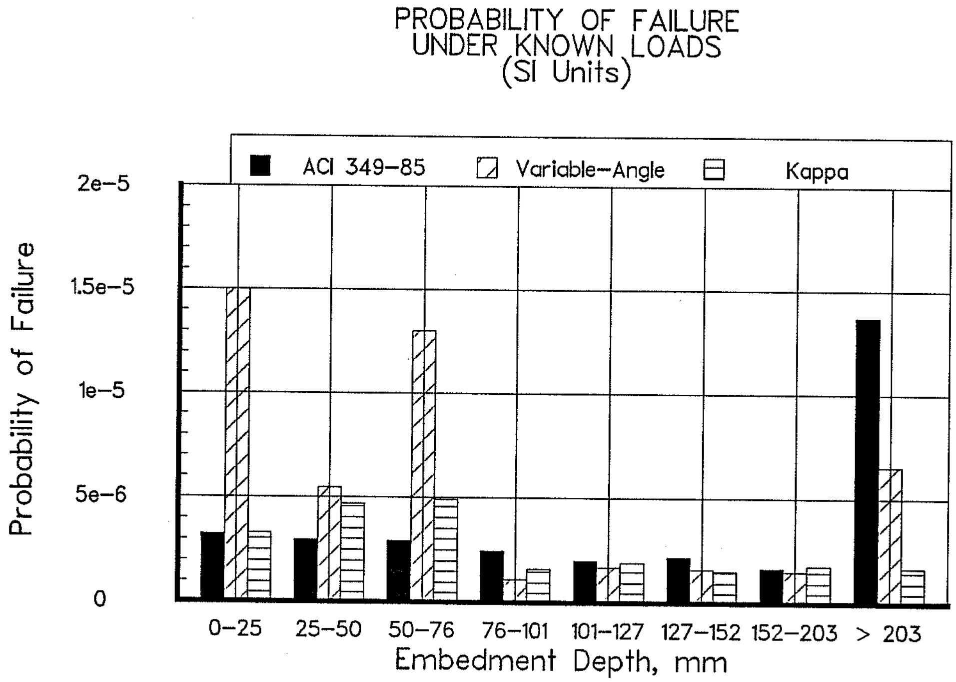

Figure 29: Comparison of Probability of Failure under Known Loads Using Each Theory (U.S. Units)

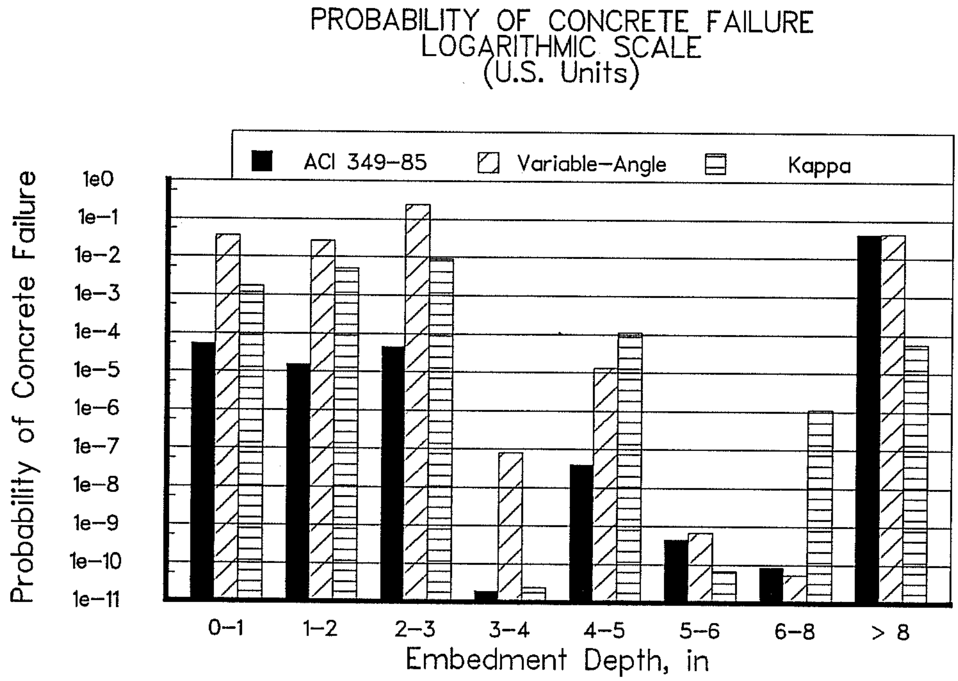

Figure 30: Comparison of Probability of Concrete Failure under Unlimited Loads Using Each Theory (U.S. Units)

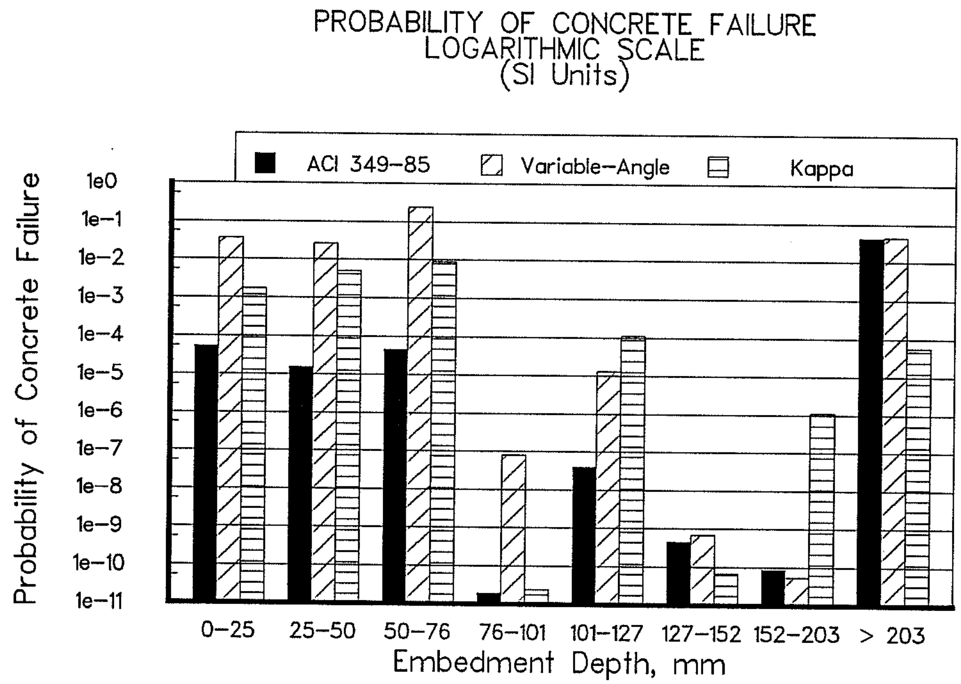

Figure 31: Comparison of Probability of Concrete Failure under Unlimited Loads Using Each Theory (SI Units)

7.0 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

7.1 Summary

The overall objective of this research was to evaluate the accuracy and suitability for design of three different theories for predicting anchor capacity as governed by concrete failure. That objective was accomplished by the following steps:

1) Approximately 800 data points, consisting of data previously surveyed by Fuchs and Breen, were available from tests on single anchors failing by formation of a concrete cone. Using common definitions and nomenclature for all variables and material properties, those data were placed on two parallel data bases, one in U.S. units and concrete cylinder strengths, the other in SI units and concrete cube strengths.

2) The data were plotted against three existing theories: the 45° cone theory of <u>ACI 349-85</u>; a variable-angle cone theory; and the Kappa theory (exponent of 1.5). The plots show concrete capacity, normalized by $\sqrt{f_c}$, as a function of embedment depth.

3) For different ranges of embedment depths, observed data were compared against those existing theories in terms of square root of the sum of the square error.

4) For different ranges of embedment depths, the probability of failure under known loads was evaluated, and also the probability of concrete failure under unlimited loads, of anchors designed according to each theory.

5) Based on those comparisons, each theory was evaluated with respect to accuracy, and also with respect to suitability for design.

7.2 Conclusions

7.2.1 Square Error:

For most embedment depths, the Kappa theory has a square error lower than that of either ACI 349-85 or the variable-angle cone theories. For embedment depths of 4 inches (101.6 mm) or less the Kappa theory has about half the error of ACI 349-85. For embedments from 4 inches (101.6 mm) to 8 inches (203.2 mm), the error is about the same. For embedments greater than 8 inches (203.2 mm), the Kappa theory again has about half of the error. Errors of all methods increase for embedment depths greater than 6 in. (150 mm).

However, this method of error analysis does not present a complete picture of the reliability of a given formula. It assigns more weight to data points located far from the values predicted by the equation under consideration. Because each data point does not contribute equally in the error analysis, some distortion is created. A few data points lying far from the curve can have as much effect as a larger number of points close to the curve. Also, this method does not distinguish systematic error from random error. Examination of Figures 4 through 9 shows that the ACI 349-85 cone theory is consistently conservative for low embedments. That fact is not revealed in comparison of square error.

7.2.2 Probability of Failure under Known Loads:

Under known loads, anchors designed using the concrete capacity provisions of any of the three theories have probabilities of failure less than 2×10^{-5} at all embedment depths. For embedment depths of 8 inches or less (203.2 mm), the probabilities of failure associated with the ACI 349-85 and the Kappa theory are of the same order of magnitude. Given the level of uncertainty discussed in Section 6.7.1, neither theory is shown to be clearly superior.

However, for embedment depths greater than 8 inches (203.2 mm), the probability of failure associated with the Kappa theory is an order of magnitude lower than that of the ACI 349-85.

All three theories for computing concrete cone capacity produce final designs with probabilities of failure between 2 x 10^{-5} and 1 x 10^{-6}. These probabilities correspond to β between 4.2 and 4.7. The Kappa theory has probabilities of failure not exceeding 5 x 10^{-6}, and is the most consistent theory for all ranges of embedment depths. The actual probability of failure of course depends on the statistical variability of the loads, which was assumed for purposes of this thesis

Current design practice for reinforced concrete structures with average consequences of failure accepts designs giving β values of 3.0 to 3.5 [18]. All three theories are more conservative than this standard. Based on the Monte Carlo analysis, it may therefore be concluded that all three concrete capacity theories give a sufficiently low probability of failure for single anchors under known loads, when used in conjunction with current ACI 349-85 load and understrength factors.

7.2.3 Probability of Concrete Failure under Unlimited Loads:

Under unlimited loads, anchors designed using the concrete capacity provision any of the three theories have probabilities of concrete failure less than 2 x 10^{-2} for all embedment depths. The only exception is the variable-angle cone theory at embedment depths of 2.01-3.0 inches (51.1-76.2 mm). Note that the probability of concrete failure under unlimited loads is independent of the load distribution.

The probability of concrete failure under unlimited loads associated with the ACI 349-85 theory is lower than that of the Kappa theory for the following ranges of embedment depths: 0.01-3.0 inches (0.3-76.2 mm); 4.01-5.0 inches (101.9-127.0 mm); 6.01-8.0 inches (152.7-203.2 mm). For embedment depths of 3.01-4.0 inches (76.5-101.6 mm) the probabilities of concrete failure under unlimited loads are approximately equal for the two theories. However for embedment depths of 5.01-6.0 inches (127.3-152.1 mm) and for embedment depths greater than 8.01 inches (203.2 mm), the probabilities of concrete failure under unlimited loads associated with the Kappa theory are lower than those of the ACI 349-85 theory. Note that the Kappa theory at its worst gives a probability of concrete failure under unlimited loads of 9.1×10^{-3} for embedment depths of 2.01-3.0 inches (51.1-76.2 mm); the theory of ACI 349-85 at its worst gives a probability of 4.2×10^{-2} for embedment depths greater than 8.01 inches (203.2 mm); and the variable-angle cone theory at its worst gives a probability of failure of 0.238 for embedment depths of 2.01-3.0 inches (51.1-76.2 mm).

For all but one embedment depth range, all three theories for computing concrete cone capacity produce final designs with probabilities of concrete failure, under unlimited loads, between 5×10^{-2} and 1×10^{-11}. The exception to this is the variable-angle cone theory applied to embedment depths between 2.01 and 3.0 inches (51.1-76.2 mm), the probability of failure is 0.238. Using the same terminology developed for (Resistance minus Load) calculations, these probabilities would correspond to β values between 1.8 and 6.5.

In assessing the significance of these probabilities of concrete failure under unlimited loads, it must be pointed out that the probability of occurrence of an unlimited load is itself very small. If anchors are designed using the procedure of ACI 349-85 Appendix B, and using the concrete capacity theory of ACI 349-85 Appendix B, the probability of concrete failure under any possible loads is less than 5%. Using the Kappa theory, the probability of concrete failure under any possible loads is less than 1%. While these probabilities is not infinitesimal, they are believed acceptable for such an extreme condition

7.2.4 Limitations of This Study:

As noted in more detail throughout, this study has the following limitations:

1) The existing concrete capacity data base has few points representing concrete failures at embedment depths exceeding 8 inches (200 mm). The lack of points in this region leads to increased statistical uncertainty regarding capacities, which in turn

causes lower values of β (higher probability of failure). Augmenting the data base with steel failures (considered as lower bounds) in this region does not help this situation. Additional data points at deep embedments are badly needed.

2) The Central Limit Theorem may not be valid for obtaining a normal distribution for the ratio of actual concrete strengths to predicted concrete strengths, for the following reasons:

- First, a normal distribution has been determined over a range of embedment depths in order to keep the number of failure points in each range of embedment depths large. For the normal distribution to be accurate, the ratio should possess similar properties over the embedment range considered. No study was performed to determine the variation of ratios from the smallest embedment depth in the range to the largest embedment depth in the range.

- The Central Limit Theorem requires a very large number of data points to determine the true normal distribution. As evident in Figs. 2 and 3, a limited number of data points are available in the deeper embedment ranges.

3) ACI 349-85 has requirements for the minimum bearing area of the anchor head. The concrete data base was not checked for compliance with those requirements.

4) Loads and resistances are assumed to be normally distributed. Safety factors computed by the analysis could increase or decrease as a result of different assumed distributions.

5) Actual concrete strength is assumed to equal the specified value. This assumption is conservative, because actual concrete strength usually exceeds that specified.

6) A single representative value is used for the ratio of specified ultimate steel strength to specified yield strength. This could be made more accurate by computing it separately for each anchor in the data base.

7) Typical field construction variations in embedment depth and edge distance are not taken into account.

8) The specimens comprising this data base involve essentially uncracked concrete. Effects of cracking would be expected to reduce the anchor capacity as governed by concrete, and would increase the probabilities of failure calculated here.

9) The analysis is intended to apply to single tensile anchors only. Safety analysis of multiple-anchor attachments can be conducted by extending the principles followed here. Care should be taken to correctly account for the effect of overlapping stress cones on the actual strength as governed by

concrete, and for the effect of failure of a single anchor on the behavior of the entire connection.

7.3 Recommendations

1) For single tension anchors with embedment depths less than 8 inches, the capacity prediction formulas of all three theories studied here (ACI 349-85, variable-angle cone, and Kappa) give satisfactory probabilities of failure against known loads, and of concrete failure against unlimited loads, when used in conjunction with the load and understrength factors of ACI 349-85. Therefore, although the Kappa theory fits most of the data better than the other two theories, all three theories can safely be used in designing single tension anchors. For single tension anchors with embedment depths greater than 8 inches the Kappa theory gives lower probability of failure under known loads, and of concrete failure under unlimited loads and therefore is recommended for design.

2) The principal attraction of the ACI 349-85 approach is its conical idealization of the failure surface. If additional accuracy were desired, this approach could be preserved by changing to a variable cone angle. In addition to representing the existing data better, this approach is also physically reasonable.

7.4 Recommendations for Further Research

1) Extend this analysis to the case of multiple tension anchors failing in concrete. Care should be taken to correctly account for the effect of overlapping stress cones on the actual strength as governed by concrete, and for the effect of failure of a single anchor on the behavior of the entire connection.

2) Seek additional data to increase the number of available results at large embedment depths. If existing results are not available, it may be necessary to conduct additional tests.

3) Extend this analysis to the following cases:

   - lightweight concrete
   - adhesive anchors

4) Extend this analysis to study the effect of the ϕ factor on the probability of failure under known loads and the probability of concrete failure under unlimited load.

APPENDIX A: ORIGINAL DATA BASE FOR CONCRETE FAILURES (SI UNITS)

At the time of compiling this thesis we did not receive permission to publish the data pertinent to Appendix A.

APPENDIX B: ORIGINAL DATA BASE FOR STEEL FAILURES (U.S. UNITS)

TEST NO.	REFERENCE	REF. NO.	TYPE	DIAMETER	LENGTH	EMBEDMENT	HEAD	CONCRETE	TYPE	EDGE DISTANCE	ULT. LOAD	MODE	REMARK
1	10	A1-1	STUD	.750	7.000	6.625	1.250	5270.	NWT	12.000	28.3	S	
2	10	A1-2	STUD	.750	7.000	6.625	1.250	5270.	NWT	12.000	28.5	S	
3	10	A1-3	STUD	.750	7.000	6.625	1.250	5270.	NWT	12.000	28.0	S	
6	10	A2-4	STUD	.750	7.000	6.625	1.250	5270.	NWT	4.000	31.5	S	
7	10	A2-5	STUD	.750	7.000	6.625	1.250	5270.	NWT	4.000	29.3	S	
8	10	A3-4	STUD	.750	7.000	6.625	1.250	5270.	NWT	6.000	29.3	S	
9	10	A3-5	STUD	.750	7.000	6.625	1.250	5270.	NWT	6.000	28.8	S	
12	10	B3-4	STUD	.750	7.000	6.625	1.250	4900.	NWT	6.000	31.5	S	
13	10	B3-5	STUD	.750	7.000	6.625	1.250	4900.	NWT	4.000	29.4	S	
19	10	C3-5	STUD	.750	7.000	6.625	1.250	5180.	NWT	6.000	27.3	S	
26	12	1/4HLA	STUD	.250	2.500	2.313	.500	2950.	NWT	12.000	3.7	S	
27	12	1/4HLB	STUD	.250	2.500	2.313	.500	2950.	NWT	12.000	3.0	S	WELD BROKE

28	12	3/8HLA	STUD	.375	3.875	3.594	.750	2950.	NWT
12.000	8.5	S							
29	12	3/8HLB	STUD	.375	3.875	3.594	.750	2950.	NWT
12.000	8.7	S							
30	12	1/2HLA	STUD	.500	5.000	4.688	1.000	2950.	NWT
12.000	12.5	S							
31	12	1/2HLB	STUD	.500	5.000	4.688	1.000	2950.	NWT
12.000	15.0	S							
32	12	5/8HLA	STUD	.625	6.250	5.938	1.250	2950.	NWT
12.000	23.7	S							
33	12	5/8HLB	STUD	.625	6.250	5.938	1.250	2950.	NWT
12.000	25.0	S							
34	12/13	5/8HLA	STUD	.625	6.250	5.938	1.125	3110.	NWT
12.000	22.0	S							
35	12/13	5/8HLB	STUD	.625	6.250	5.938	1.125	3110.	NWT
12.000	25.0	S							
36	12	3/4HLA	STUD	.750	7.500	7.125	1.500	2950.	NWT
12.000	33.0	S	WELD BROKE						
37	12	3/4HLB	STUD	.750	7.500	7.125	1.500	2950.	NWT
12.000	34.5	S	WELD BROKE						
38	12	3/4HLA	STUD	.750	7.500	7.125	1.250	3110.	NWT
12.000	33.2	S	MACH.BROKE						
39	12	3/4HLB	STUD	.750	7.500	7.125	1.250	3110.	NWT
12.000	38.1	S							
40	12	7/8HLA	STUD	.875	7.500	7.125	1.750	2950.	NWT
12.000	50.5	S	WELD BROKE						
41	12	7/8HLB	STUD	.875	7.500	7.125	1.750	2950.	NWT
12.000	42.5	S	MACH.BROKE						

42	12		1/2THA	STUD	.500	5.000	4.688	.875	3110.	NWT
12.000		14.5	S	THR.STUD						
43	12		1/2THB	STUD	.500	5.000	4.688	.875	3110.	NWT
12.000		16.8	S	THR.STUD						
44	12		5/8THA	STUD	.625	6.250	5.938	1.125	3110.	NWT
12.000		23.1	S	THR.STUD						
45	12		5/8THB	STUD	.625	6.250	5.938	1.125	3110.	NWT
12.000		24.3	S	THR.STUD						
46	12		3/4THA	STUD	.750	7.500	7.125	1.250	3110.	NWT
12.000		35.0	S	THR.STUD						
47	12		3/4THB	STUD	.750	7.500	7.125	1.250	3110.	NWT
12.000		35.0	S	MACH.BROKE						
51	7			A490	1.000	12.600	12.000	1.625	4300.	NWT
18.000		116.0	S							
52	7			A490	1.000	14.700	14.100	1.625	4245.	NWT
18.000		118.0	S							
53	7			A490	1.000	16.800	16.200	1.625	4200.	NWT
18.000		118.0	S							
54	7			A307	.750	6.000	5.500	1.125	5050.	NWT
5.000		26.1	S							
55	7			A307	.750	6.000	5.500	1.125	5050.	NWT
6.000		26.2	S							
57	7			A307	.750	7.000	6.500	1.125	4000.	NWT
2.000		25.4	S							
58	7			A307	.750	7.000	6.500	1.125	5050.	NWT
4.000		26.3	S							
59	7			A307	.750	7.000	6.500	1.125	5050.	NWT
5.000		29.6	S							

60	7	A307	.750	8.000	7.500	1.125	3500.	NWT	2.000	23.2	S	
61	7	A307	.750	8.000	7.500	1.125	3500.	NWT	4.000	24.4	S	
67	7	A307	.750	5.000	4.500	1.125	5500.	NWT	3.750	29.9	S	GROUTED
68	7/14	A307	.750	6.000	5.500	1.125	5500.	NWT	2.500	25.4	S	GROUTED
78	7	A307	.750	5.000	4.500	1.125	4635.	NWT	5.000	21.0	S	
82	7	A307	.750	6.000	5.500	1.125	5050.	NWT	3.000	26.0	S	
83	7	A307	.750	6.000	5.500	1.125	5050.	NWT	4.000	26.1	S	
84	7	A307	.750	6.000	5.500	1.125	5500.	NWT	3.000	26.0	S	GROUTED
86	7	A307	.750	7.000	6.500	1.125	5500.	NWT	2.750	30.4	S	GROUTED
87	7	A307	.750	8.000	7.500	1.125	5500.	NWT	2.750	29.3	S	GROUTED
88	7	A307	.750	8.000	7.500	1.125	5500.	NWT	2.750	27.7	S	GROUTED

REFERENCES FOR APPENDIX B

7. Cannon, R. W., Burdette, E. G., and Funk, R. R., "Anchorage to Concrete," Tennessee Valley Authority, Knoxville, Tennessee, Dec. 1975.

10. McMackin, P. J., Slutter, R. G., and Fisher, J. W., "Headed Steel Anchors under Combined Loading," AISC Engineeering Journal, Second Quarter, Apr. 1973, pp. 43-52.

12. "Concrete Anchor Design Data," Manual No. 21, Nelson Stud Welding Company, Lorain, Aug. 1961.

13. "Nelson Stud Project No. 87," Report No. 1960-16: Concrete Arctests No. 5, Nelson Stud Welding Company, Lorain, 1960.

14. Bailey, John W., and Burdette, Edwin G., "Edge Effects on Anchorage to Concrete," Civil Engineering Research Series No. 31, The University of Tennessee, Knoxville, Aug. 1977, 120 pp.

APPENDIX C: SPREADSHEET DATA BASE FOR CONCRETE FAILURE (U.S.UNITS)

	A	B	C	D	E	F	G	H
1	Anchor	Concrete	Actual	Embedment	Normalized	ACI 349-85	Kappa	VCA
2	Diameter	Strength	Load	Depth	Actual	Equation	Equation	Equation
3	in	fc	Capacity	in	Load	sqrt(lb)*in	sqrt(lb)*in	sqrt(lb)*in
4		psi	Kips		Capacity			
5					sqrt(lb)*in			
6								
7	Embedment Depth * 2 > Edge Distance							
8	.768	5297	30.081	7.622	413.3	803.6	846.8	803.6
9	.768	5297	31.497	7.622	432.8	803.6	846.8	803.6
10	.768	4867	19.897	3.500	285.2	187.7	263.5	260.6
11	.874	4892	42.986	7.622	614.6	813.8	846.8	813.8
12	1.000	4314	97.999	9.898	1,492.0	1,355.4	1,253.0	1,355.4
13	Embedment Depth * 2 <= Edge Distance							
14	.768	2999	11.983	3.626	218.8	200.2	277.8	270.5
15	.768	5174	18.480	3.626	256.9	200.2	277.8	270.5
16	.768	3110	15.985	2.500	286.7	102.7	159.1	176.0
17	.768	2999	10.994	3.626	200.8	200.2	277.8	270.5
18	.768	5174	18.480	3.626	256.9	200.2	277.8	270.5
19	.768	3110	25.382	3.500	455.2	187.7	263.5	260.6
20	.768	2999	13.984	3.626	255.4	200.2	277.8	270.5
21	.768	5174	17.289	3.626	240.3	200.2	277.8	270.5
22	1.000	3331	14.006	2.992	242.7	150.1	208.3	229.9
23	1.000	3110	22.505	5.000	403.6	377.0	449.9	377.0
24	1.000	3097	26.394	5.000	474.3	377.0	449.9	377.0
25	.500	2987	7.801	3.689	142.7	194.2	285.1	260.8
26	.626	2987	14.006	3.437	256.3	175.5	256.4	248.2
27	.626	2987	12.410	3.437	227.1	175.5	256.4	248.2
28	.626	2987	9.397	3.437	172.0	175.5	256.4	248.2
29	.748	2987	14.006	3.437	256.3	180.8	256.4	254.5
30	.748	2987	10.791	3.500	197.5	186.8	263.5	259.5
31	.748	2987	10.791	3.500	197.5	186.8	263.5	259.5
32	.748	2987	12.410	3.500	227.1	186.8	263.5	259.5
33	.748	2987	11.691	3.500	213.9	186.8	263.5	259.5
34	.874	2987	10.791	3.500	197.5	192.4	263.5	266.2
35	.874	2987	12.410	3.500	227.1	192.4	263.5	266.2
36	.874	2987	14.006	3.500	256.3	192.4	263.5	266.2
37	k3usa.pri							

	A	B	C	D	E	F	G	H
1	Anchor	Concrete	Actual	Embedment	Normalized	ACI 349-85	Kappa	VCA
2	Diameter	Strength	Load	Depth	Actual	Equation	Equation	Equation
3	in	fc	Capacity	in	Load	sqrt(lb)*in	sqrt(lb)*in	sqrt(lb)*in
4		psi	Kips		Capacity			
5					sqrt(lb)*in			
6								
7	Embedment Depth * 2 > Edge Distance							
8	.374	1696	2.946	1.689	71.5	43.8	88.3	92.2
9	.374	1696	3.125	1.689	75.9	43.8	88.3	92.2
10	.374	1696	3.304	1.689	80.2	43.8	88.3	92.2
11	.374	3491	4.509	1.689	76.3	43.8	88.3	92.2
12	.374	3491	4.732	1.689	80.1	43.8	88.3	92.2
13	.374	3491	4.732	1.689	80.1	43.8	88.3	92.2
14	.626	1696	7.143	2.642	173.4	108.5	172.8	182.2
15	.626	1696	7.321	2.642	177.8	108.5	172.8	182.2
16	.626	1696	7.411	2.642	179.9	108.5	172.8	182.2
17	.626	3491	10.625	2.642	179.8	108.5	172.8	182.2
18	.626	3491	11.071	2.642	187.4	108.5	172.8	182.2
19	.626	3491	11.786	2.642	199.5	108.5	172.8	182.2
20	.874	1696	10.313	3.555	250.4	197.9	269.7	270.6
21	.874	1696	10.357	3.555	251.5	197.9	269.7	270.6
22	.874	1696	10.670	3.555	259.1	197.9	269.7	270.6
23	.874	3491	14.911	3.555	252.4	197.9	269.7	270.6
24	.874	3491	15.714	3.555	266.0	197.9	269.7	270.6
25	.874	3491	15.893	3.555	269.0	197.9	269.7	270.6
26	1.969	3405	193.817	20.669	3,321.7	5,879.9	3,781.3	5,879.9
27	1.969	2348	163.237	20.669	3,369.1	5,879.9	3,781.3	5,879.9
28	1.969	2348	171.897	20.669	3,547.8	5,879.9	3,781.3	5,879.9
29	1.969	2348	187.924	20.669	3,878.6	5,879.9	3,781.3	5,879.9
30	1.969	2913	183.304	20.669	3,488.9	5,879.9	3,781.3	5,879.9
31	1.969	2913	192.031	20.669	3,558.0	5,879.9	3,781.3	5,879.9
32	1.969	2348	192.344	20.669	3,969.8	5,879.9	3,781.3	5,879.9
33	1.969	2913	197.321	20.669	3,656.0	5,879.9	3,781.3	5,879.9
34	1.969	2913	204.821	20.669	3,795.0	5,879.9	3,781.3	5,879.9
35	1.969	3405	197.589	20.669	3,386.3	5,879.9	3,781.3	5,879.9
36	1.969	2348	187.188	20.669	3,863.4	5,879.9	3,781.3	5,879.9
37	1.969	2913	190.268	20.669	3,525.3	5,879.9	3,781.3	5,879.9
38	1.969	2913	205.201	20.669	3,802.0	5,879.9	3,781.3	5,879.9
39	1.969	2348	171.250	20.669	3,534.5	5,879.9	3,781.3	5,879.9
40	Embedment Depth * 2 <= Edge Distance							
41	.374	3429	3.393	1.689	57.9	43.8	88.3	92.2

	A	B	C	D	E	F	G	H
42	.374	3491	3.571	1.689	60.4	43.8	88.3	92.2
43	.374	3085	4.063	1.689	73.1	43.8	88.3	92.2
44	.374	3085	4.286	1.689	77.2	43.8	88.3	92.2
45	.374	3429	4.598	1.689	78.5	43.8	88.3	92.2
46	.374	3429	4.598	1.689	78.5	43.8	88.3	92.2
47	.374	3085	4.643	1.689	83.6	43.8	88.3	92.2
48	.374	3085	4.643	1.689	83.6	43.8	88.3	92.2
49	.374	3429	4.688	1.689	80.0	43.8	88.3	92.2
50	.374	3429	4.732	1.689	80.8	43.8	88.3	92.2
51	.374	3085	4.911	1.689	88.4	43.8	88.3	92.2
52	.374	3429	5.045	1.689	86.1	43.8	88.3	92.2
53	.374	4326	5.134	1.689	78.1	43.8	88.3	92.2
54	.374	3491	5.357	1.689	90.7	43.8	88.3	92.2
55	.374	3085	5.536	1.689	99.7	43.8	88.3	92.2
56	.374	4486	5.580	1.689	83.3	43.8	88.3	92.2
57	.374	4486	5.580	1.689	83.3	43.8	88.3	92.2
58	.374	4326	5.692	1.689	86.5	43.8	88.3	92.2
59	.374	4486	6.027	1.689	90.0	43.8	88.3	92.2
60	.374	4326	6.027	1.689	91.6	43.8	88.3	92.2
61	.374	4326	6.027	1.689	91.6	43.8	88.3	92.2
62	.374	4326	6.138	1.689	93.3	43.8	88.3	92.2
63	.374	4326	6.585	1.689	100.1	43.8	88.3	92.2
64	.874	3749	6.808	2.571	111.2	111.3	165.9	186.9
65	.874	3749	9.241	2.571	150.9	111.3	165.9	186.9
66	.874	3749	9.464	2.571	154.6	111.3	165.9	186.9
67	.626	2790	8.259	2.642	156.4	108.5	172.8	182.2
68	.626	2790	9.040	2.642	171.1	108.5	172.8	182.2
69	.626	2790	9.375	2.642	177.5	108.5	172.8	182.2
70	.626	2790	9.710	2.642	183.8	108.5	172.8	182.2
71	.626	2790	9.821	2.642	185.9	108.5	172.8	182.2
72	.626	2790	11.049	2.642	209.2	108.5	172.8	182.2
73	.626	1401	6.473	2.642	172.9	108.5	172.8	182.2
74	.626	1401	7.143	2.642	190.8	108.5	172.8	182.2
75	.626	1401	8.259	2.642	220.6	108.5	172.8	182.2
76	.626	2790	8.371	2.642	158.5	108.5	172.8	182.2
77	.626	2790	8.705	2.642	164.8	108.5	172.8	182.2
78	.626	2790	8.929	2.642	169.0	108.5	172.8	182.2
79	.626	2790	8.929	2.642	169.0	108.5	172.8	182.2
80	.626	2790	9.375	2.642	177.5	108.5	172.8	182.2
81	.626	2790	10.045	2.642	190.2	108.5	172.8	182.2
82	.626	5703	12.054	2.642	159.6	108.5	172.8	182.2

	A	B	C	D	E	F	G	H
83	.626	5703	12.500	2.642	165.5	108.5	172.8	182.2
84	.626	5703	12.723	2.642	168.5	108.5	172.8	182.2
85	.626	4585	12.835	2.642	189.6	108.5	172.8	182.2
86	.626	4585	12.946	2.642	191.2	108.5	172.8	182.2
87	.626	4585	13.170	2.642	194.5	108.5	172.8	182.2
88	.626	5703	13.170	2.642	174.4	108.5	172.8	182.2
89	.626	5703	13.393	2.642	177.3	108.5	172.8	182.2
90	.626	5703	13.616	2.642	180.3	108.5	172.8	182.2
91	.874	3749	15.446	3.555	252.3	197.9	269.7	270.6
92	.874	3749	15.893	3.555	259.6	197.9	269.7	270.6
93	.874	3749	16.964	3.555	277.1	197.9	269.7	270.6
94	.874	5703	17.522	3.555	232.0	197.9	269.7	270.6
95	.874	5703	17.857	3.555	236.5	197.9	269.7	270.6
96	.874	5703	18.192	3.555	240.9	197.9	269.7	270.6
97	.874	5703	18.304	3.555	242.4	197.9	269.7	270.6
98	.874	5703	18.638	3.555	246.8	197.9	269.7	270.6
99	.874	5703	20.536	3.555	271.9	197.9	269.7	270.6
100	.874	3429	13.839	3.555	236.3	197.9	269.7	270.6
101	.874	3429	14.286	3.555	244.0	197.9	269.7	270.6
102	.874	3429	14.464	3.555	247.0	197.9	269.7	270.6
103	.874	3429	14.643	3.555	250.1	197.9	269.7	270.6
104	.874	3085	15.089	3.555	271.7	197.9	269.7	270.6
105	.874	3429	15.268	3.555	260.7	197.9	269.7	270.6
106	.874	3429	15.268	3.555	260.7	197.9	269.7	270.6
107	.874	3491	15.402	3.555	260.7	197.9	269.7	270.6
108	.874	4326	16.295	3.555	247.7	197.9	269.7	270.6
109	.874	3085	16.339	3.555	294.2	197.9	269.7	270.6
110	.874	3491	16.518	3.555	279.6	197.9	269.7	270.6
111	.874	4326	16.518	3.555	251.1	197.9	269.7	270.6
112	.874	4326	16.518	3.555	251.1	197.9	269.7	270.6
113	.874	3085	16.875	3.555	303.8	197.9	269.7	270.6
114	.874	4486	16.964	3.555	253.3	197.9	269.7	270.6
115	.874	3491	16.964	3.555	287.1	197.9	269.7	270.6
116	.874	3085	16.964	3.555	305.4	197.9	269.7	270.6
117	.874	3749	17.143	3.555	280.0	197.9	269.7	270.6
118	.874	3749	17.321	3.555	282.9	197.9	269.7	270.6
119	.874	4326	17.411	3.555	264.7	197.9	269.7	270.6
120	.874	3085	17.589	3.555	316.7	197.9	269.7	270.6
121	.874	3085	17.679	3.555	318.3	197.9	269.7	270.6
122	.874	4326	17.857	3.555	271.5	197.9	269.7	270.6
123	.874	3749	17.857	3.555	291.7	197.9	269.7	270.6

	A	B	C	D	E	F	G	H
124	.874	4486	18.304	3.555	273.3	197.9	269.7	270.6
125	.874	4486	18.415	3.555	274.9	197.9	269.7	270.6
126	.874	4326	18.973	3.555	288.5	197.9	269.7	270.6
127	.626	1905	9.643	3.626	220.9	193.7	277.8	262.9
128	.626	1905	10.625	3.626	243.4	193.7	277.8	262.9
129	.626	1905	11.161	3.626	255.7	193.7	277.8	262.9
130	.626	1401	9.375	3.626	250.5	193.7	277.8	262.9
131	.626	1401	10.268	3.626	274.3	193.7	277.8	262.9
132	.626	1401	11.384	3.626	304.1	193.7	277.8	262.9
133	.626	4585	19.643	3.626	290.1	193.7	277.8	262.9
134	.626	4585	20.313	3.626	300.0	193.7	277.8	262.9
135	.626	4585	21.875	3.626	323.1	193.7	277.8	262.9
136	.748	2495	16.071	4.539	321.7	301.6	389.2	333.9
137	.748	2495	16.071	4.539	321.7	301.6	389.2	333.9
138	.748	2495	19.464	4.539	389.7	301.6	389.2	333.9
139	.874	3749	21.071	4.539	344.2	308.8	389.2	341.5
140	.874	3749	21.786	4.539	355.8	308.8	389.2	341.5
141	.874	3749	22.500	4.539	367.5	308.8	389.2	341.5
142	.866	3405	21.696	5.118	371.8	384.9	465.9	384.9
143	.874	2311	19.107	5.524	397.5	444.1	522.4	444.1
144	.866	3048	38.728	7.283	701.5	745.9	791.0	745.9
145	.866	2729	33.616	7.283	643.5	745.9	791.0	745.9
146	.866	2729	33.862	7.283	648.2	745.9	791.0	745.9
147	.866	2729	32.500	7.283	622.2	745.9	791.0	745.9
148	.866	3331	41.741	7.283	723.2	745.9	791.0	745.9
149	.866	3331	44.643	7.283	773.5	745.9	791.0	745.9
150	.866	3466	46.205	7.283	784.8	745.9	791.0	745.9
151	.866	3650	49.777	7.283	823.9	745.9	791.0	745.9
152	.866	3638	49.777	7.283	825.3	745.9	791.0	745.9
153	.866	4068	49.777	7.283	780.4	745.9	791.0	745.9
154	.866	3331	50.446	7.283	874.1	745.9	791.0	745.9
155	.866	3638	50.893	7.283	843.8	745.9	791.0	745.9
156	.866	3417	51.339	7.283	878.3	745.9	791.0	745.9
157	.866	3331	52.679	7.283	912.8	745.9	791.0	745.9
158	.866	3650	53.348	7.283	883.0	745.9	791.0	745.9
159	.866	3687	53.795	7.283	885.9	745.9	791.0	745.9
160	.866	3638	57.813	7.283	958.5	745.9	791.0	745.9
161	.866	2729	33.125	7.283	634.1	745.9	791.0	745.9
162	.866	1917	28.571	7.283	652.5	745.9	791.0	745.9
163	.866	2016	28.661	7.283	638.4	745.9	791.0	745.9
164	.866	2016	30.134	7.283	671.2	745.9	791.0	745.9

	A	B	C	D	E	F	G	H
165	.866	1917	30.938	7.283	706.5	745.9	791.0	745.9
166	.866	2483	40.513	7.283	813.1	745.9	791.0	745.9
167	.866	2483	40.692	7.283	816.7	745.9	791.0	745.9
168	.866	2446	40.781	7.283	824.6	745.9	791.0	745.9
169	.866	2446	44.866	7.283	907.2	745.9	791.0	745.9
170	.866	2729	31.540	7.283	603.8	745.9	791.0	745.9
171	1.181	5408	91.920	9.843	1,249.9	1,363.5	1,242.6	1,363.5
172	1.181	4179	91.987	9.843	1,423.0	1,363.5	1,242.6	1,363.5
173	1.181	5408	92.656	9.843	1,260.0	1,363.5	1,242.6	1,363.5
174	1.181	5408	97.232	9.843	1,322.2	1,363.5	1,242.6	1,363.5
175	1.181	4179	97.411	9.843	1,506.9	1,363.5	1,242.6	1,363.5
176	1.181	8714	111.384	9.843	1,193.2	1,363.5	1,242.6	1,363.5
177	1.181	8653	119.420	9.843	1,283.8	1,363.5	1,242.6	1,363.5
178	1.181	8714	124.799	9.843	1,336.9	1,363.5	1,242.6	1,363.5
179	1.181	8837	137.835	9.843	1,466.2	1,363.5	1,242.6	1,363.5
180	1.181	8837	138.505	9.843	1,473.4	1,363.5	1,242.6	1,363.5
181	1.181	8837	139.330	9.843	1,482.1	1,363.5	1,242.6	1,363.5
182	1.181	2729	65.692	10.236	1,257.6	1,468.6	1,317.9	1,468.6
183	1.181	2729	67.433	10.236	1,290.9	1,468.6	1,317.9	1,468.6
184	1.181	2729	58.549	10.236	1,120.9	1,468.6	1,317.9	1,468.6
185	1.181	2729	59.509	10.236	1,139.2	1,468.6	1,317.9	1,468.6
186	1.181	2729	60.424	10.236	1,156.8	1,468.6	1,317.9	1,468.6
187	1.181	2729	61.071	10.236	1,169.1	1,468.6	1,317.9	1,468.6
188	1.181	2729	62.054	10.236	1,188.0	1,468.6	1,317.9	1,468.6
189	1.181	2729	63.817	10.236	1,221.7	1,468.6	1,317.9	1,468.6
190	1.181	3405	64.732	10.236	1,109.4	1,468.6	1,317.9	1,468.6
191	1.575	2729	100.580	13.976	1,925.5	2,731.3	2,102.6	2,731.3
192	1.575	2729	100.893	13.976	1,931.5	2,731.3	2,102.6	2,731.3
193	1.575	2729	103.951	13.976	1,990.0	2,731.3	2,102.6	2,731.3
194	1.575	2729	104.464	13.976	1,999.9	2,731.3	2,102.6	2,731.3
195	1.575	2729	107.076	13.976	2,049.9	2,731.3	2,102.6	2,731.3
196	1.575	2729	105.022	13.976	2,010.5	2,731.3	2,102.6	2,731.3
197	1.575	2729	106.161	13.976	2,032.3	2,731.3	2,102.6	2,731.3
198	1.575	2729	94.107	13.976	1,801.6	2,731.3	2,102.6	2,731.3
199	1.575	2729	88.170	13.976	1,687.9	2,731.3	2,102.6	2,731.3
200	1.575	2729	98.058	13.976	1,877.2	2,731.3	2,102.6	2,731.3
201	k4ger.cal							

	A	B	C	D	E	F	G	H
1	Anchor	Concrete	Actual	Embedment	Normalized	ACI 349-85	Kappa	VCA
2	Diameter	Strength	Load	Depth	Actual	Equation	Equation	Equation
3	in	fc	Capacity	in	Load	sqrt(lb)*in	sqrt(lb)*in	sqrt(lb)*in
4		psi	kips		Capacity			
5					sqrt(lb)*in			
6								
7	Embedment Depth * 2 > Edge Distance							
8	.236	3970	2.046	1.102	32.5	18.5	40.6	45.2
9	.472	3073	2.900	1.220	52.3	26.0	47.3	58.9
10	.472	3073	3.103	1.457	56.0	35.3	61.6	76.8
11	.236	2950	3.215	1.457	59.2	31.0	61.6	70.1
12	.315	2827	3.372	1.575	63.4	37.4	69.3	81.4
13	.315	7252	5.890	1.575	69.2	37.4	69.3	81.4
14	.315	3196	3.597	1.575	63.6	37.4	69.3	81.4
15	.315	3663	4.069	1.575	67.2	37.4	69.3	81.4
16	.315	5617	5.171	1.575	69.0	37.4	69.3	81.4
17	.315	3196	4.766	1.969	84.3	56.5	96.8	112.9
18	.669	3073	7.014	2.087	126.5	72.3	105.6	135.8
19	.394	3503	8.386	2.362	141.7	81.8	127.3	148.6
20	.394	8272	12.298	2.362	135.2	81.8	127.3	148.6
21	.315	3134	6.745	2.362	120.5	79.5	127.3	145.4
22	.472	2950	9.532	3.150	175.5	143.4	195.9	217.4
23	.472	2950	10.229	3.150	188.3	143.4	195.9	217.4
24	.984	3073	13.602	3.268	245.4	174.6	207.0	252.7
25	.630	4302	18.480	3.937	281.8	225.9	273.8	286.2
26	.630	4634	27.181	4.921	399.3	343.3	382.7	349.4
27	.787	3478	41.187	6.693	698.4	629.1	606.9	629.1
28	.945	5162	58.183	7.874	809.8	872.6	774.4	872.6
29	.945	4081	52.900	8.661	828.1	1,045.6	893.5	1,045.6
30	Embedment Depth * 2 <= Edge Distance							
31	.236	2704	1.326	.693	25.5	8.1	20.2	21.1
32	.236	6023	1.844	.693	23.8	8.1	20.2	21.1
33	.315	3073	1.484	.984	26.8	16.1	34.2	39.2
34	.315	3073	1.844	.984	33.3	16.1	34.2	39.2
35	.315	3073	1.686	.984	30.4	16.1	34.2	39.2
36	.236	2704	2.248	.984	43.2	15.1	34.2	37.6
37	.236	6023	3.462	.984	44.6	15.1	34.2	37.6
38	.315	1377	.742	.984	20.0	16.1	34.2	39.2
39	.315	2741	1.259	.984	24.0	16.1	34.2	39.2
40	.315	1536	1.371	.984	35.0	16.1	34.2	39.2
41	.315	3024	1.619	.984	29.4	16.1	34.2	39.2

	A	B	C	D	E	F	G	H
42	.315	3638	2.293	.984	38.0	16.1	34.2	39.2
43	.315	5248	2.383	.984	32.9	16.1	34.2	39.2
44	.315	6231	3.035	.984	38.4	16.1	34.2	39.2
45	.354	2704	2.248	1.047	43.2	18.4	37.6	44.1
46	.354	6023	3.103	1.047	40.0	18.4	37.6	44.1
47	.236	2016	1.416	1.102	31.5	18.5	40.6	45.2
48	.236	2520	1.821	1.102	36.3	18.5	40.6	45.2
49	.394	5629	2.540	1.102	33.9	20.7	40.6	48.7
50	.236	7313	2.788	1.102	32.6	18.5	40.6	45.2
51	.394	6981	3.440	1.102	41.2	20.7	40.6	48.7
52	.394	3073	2.158	1.181	38.9	23.4	45.0	54.2
53	.394	3073	2.540	1.181	45.8	23.4	45.0	54.2
54	.394	3073	2.563	1.181	46.2	23.4	45.0	54.2
55	.394	1610	1.259	1.181	31.4	23.4	45.0	54.2
56	.394	2783	2.203	1.181	42.0	23.4	45.0	54.2
57	.394	2532	2.248	1.181	44.7	23.4	45.0	54.2
58	.394	2274	2.653	1.181	55.6	23.4	45.0	54.2
59	.394	3245	3.080	1.181	54.1	23.4	45.0	54.2
60	.394	4806	4.024	1.181	58.1	23.4	45.0	54.2
61	.472	1340	1.506	1.220	41.2	26.0	47.3	58.9
62	.472	1758	1.978	1.220	47.2	26.0	47.3	58.9
63	.472	2323	2.023	1.220	42.0	26.0	47.3	58.9
64	.472	3282	2.428	1.220	42.4	26.0	47.3	58.9
65	.472	3675	3.395	1.220	56.0	26.0	47.3	58.9
66	.472	5297	4.384	1.220	60.2	26.0	47.3	58.9
67	.472	7842	4.564	1.220	51.5	26.0	47.3	58.9
68	.236	2851	2.046	1.260	38.3	23.7	49.6	55.9
69	.394	2950	2.900	1.417	53.4	32.3	59.1	71.6
70	.394	7252	4.924	1.417	57.8	32.3	59.1	71.6
71	.394	2950	3.620	1.417	66.6	32.3	59.1	71.6
72	.394	7252	4.991	1.417	58.6	32.3	59.1	71.6
73	.394	2950	3.799	1.417	70.0	32.3	59.1	71.6
74	.394	7252	5.531	1.417	64.9	32.3	59.1	71.6
75	.394	2950	4.137	1.417	76.2	32.3	59.1	71.6
76	.394	7252	4.946	1.417	58.1	32.3	59.1	71.6
77	.394	1598	2.338	1.417	58.5	32.3	59.1	71.6
78	.394	4671	3.799	1.417	55.6	32.3	59.1	71.6
79	.394	3441	3.822	1.417	65.1	32.3	59.1	71.6
80	.394	6760	5.283	1.417	64.3	32.3	59.1	71.6
81	.315	3073	3.372	1.437	60.8	31.6	60.4	70.9
82	.315	6883	5.013	1.437	60.4	31.6	60.4	70.9

	A	B	C	D	E	F	G	H
83	.315	3073	3.485	1.437	62.9	31.6	60.4	70.9
84	.315	6883	5.441	1.437	65.6	31.6	60.4	70.9
85	.315	3073	3.957	1.437	71.4	31.6	60.4	70.9
86	.315	6883	5.733	1.437	69.1	31.6	60.4	70.9
87	.315	3073	3.934	1.437	71.0	31.6	60.4	70.9
88	.315	6883	5.778	1.437	69.6	31.6	60.4	70.9
89	.315	1598	2.945	1.437	73.7	31.6	60.4	70.9
90	.315	3441	3.642	1.437	62.1	31.6	60.4	70.9
91	.315	4671	3.665	1.437	53.6	31.6	60.4	70.9
92	.315	6883	4.564	1.437	55.0	31.6	60.4	70.9
93	.236	2950	3.125	1.457	57.5	31.0	61.6	70.1
94	.236	2950	3.620	1.457	66.6	31.0	61.6	70.1
95	.236	2950	3.395	1.457	62.5	31.0	61.6	70.1
96	.472	3073	3.552	1.457	64.1	35.3	61.6	76.8
97	.236	2950	3.799	1.457	70.0	31.0	61.6	70.1
98	.472	3073	3.642	1.457	65.7	35.3	61.6	76.8
99	.236	2950	3.844	1.457	70.8	31.0	61.6	70.1
100	.472	3073	3.777	1.457	68.1	35.3	61.6	76.8
101	.236	1598	2.136	1.457	53.4	31.0	61.6	70.1
102	.472	1536	2.630	1.457	67.1	35.3	61.6	76.8
103	.551	2815	3.260	1.457	61.4	36.8	61.6	79.0
104	.236	3441	3.530	1.457	60.2	31.0	61.6	70.1
105	.236	4671	3.620	1.457	53.0	31.0	61.6	70.1
106	.472	3245	4.406	1.457	77.4	35.3	61.6	76.8
107	.551	7805	5.171	1.457	58.5	36.8	61.6	79.0
108	.394	3073	3.058	1.496	55.2	35.5	64.1	77.6
109	.394	3146	2.563	1.496	45.7	35.5	64.1	77.6
110	.551	1696	2.675	1.496	65.0	38.5	64.1	82.2
111	.551	2520	2.923	1.496	58.2	38.5	64.1	82.2
112	.551	3073	3.530	1.496	63.7	38.5	64.1	82.2
113	.551	3343	4.317	1.496	74.7	38.5	64.1	82.2
114	.551	4425	4.474	1.496	67.3	38.5	64.1	82.2
115	.551	5420	5.193	1.496	70.5	38.5	64.1	82.2
116	.551	7756	5.823	1.496	66.1	38.5	64.1	82.2
117	.512	2544	3.305	1.535	65.5	39.5	66.7	84.2
118	.512	2950	3.934	1.535	72.4	39.5	66.7	84.2
119	.512	6023	6.272	1.535	80.8	39.5	66.7	84.2
120	.354	2704	4.451	1.575	85.6	38.2	69.3	82.6
121	.354	6023	7.666	1.575	98.8	38.2	69.3	82.6
122	.394	1930	2.383	1.575	54.2	39.0	69.3	83.8
123	.394	2692	2.428	1.575	46.8	39.0	69.3	83.8

	A	B	C	D	E	F	G	H
124	.472	1610	2.585	1.575	64.4	40.5	69.3	86.1
125	.472	2470	2.968	1.575	59.7	40.5	69.3	86.1
126	.472	2532	3.350	1.575	66.6	40.5	69.3	86.1
127	.394	4904	3.485	1.575	49.8	39.0	69.3	83.8
128	.472	2532	3.665	1.575	72.8	40.5	69.3	86.1
129	.472	3749	3.777	1.575	61.7	40.5	69.3	86.1
130	.472	3220	4.137	1.575	72.9	40.5	69.3	86.1
131	.472	2679	4.182	1.575	80.8	40.5	69.3	86.1
132	.472	3220	4.451	1.575	78.4	40.5	69.3	86.1
133	.472	5789	4.811	1.575	63.2	40.5	69.3	86.1
134	.472	8849	5.531	1.575	58.8	40.5	69.3	86.1
135	.315	3700	5.148	1.575	84.6	37.4	69.3	81.4
136	.315	3700	5.306	1.575	87.2	37.4	69.3	81.4
137	.315	7878	5.396	1.575	60.8	37.4	69.3	81.4
138	.315	7878	6.160	1.575	69.4	37.4	69.3	81.4
139	.236	3810	3.260	1.575	52.8	35.8	69.3	79.0
140	.276	4302	5.688	1.575	86.7	36.6	69.3	80.2
141	.276	4302	5.688	1.575	86.7	36.6	69.3	80.2
142	.315	3257	3.799	1.575	66.6	37.4	69.3	81.4
143	.315	3257	3.799	1.575	66.6	37.4	69.3	81.4
144	.315	3614	4.092	1.575	68.1	37.4	69.3	81.4
145	.315	3614	4.092	1.575	68.1	37.4	69.3	81.4
146	.315	2851	2.833	1.654	53.0	40.9	74.5	87.5
147	.394	2692	2.900	1.732	55.9	46.3	79.9	96.3
148	.394	3380	3.327	1.732	57.2	46.3	79.9	96.3
149	.630	2643	4.092	1.772	79.6	53.5	82.7	107.3
150	.669	2544	4.766	1.772	94.5	54.3	82.7	108.6
151	.669	2950	6.205	1.772	114.2	54.3	82.7	108.6
152	.669	6023	9.060	1.772	116.7	54.3	82.7	108.6
153	.591	3073	5.013	1.811	90.4	54.7	85.4	109.3
154	.591	3073	6.228	1.811	112.3	54.7	85.4	109.3
155	.591	3073	6.003	1.811	108.3	54.7	85.4	109.3
156	.591	3073	4.946	1.811	89.2	54.7	85.4	109.3
157	.591	3073	6.250	1.811	112.8	54.7	85.4	109.3
158	.591	1524	3.867	1.811	99.1	54.7	85.4	109.3
159	.591	3540	6.879	1.811	115.6	54.7	85.4	109.3
160	.591	4462	8.094	1.811	121.2	54.7	85.4	109.3
161	.591	6084	9.735	1.811	124.8	54.7	85.4	109.3
162	.236	1807	2.203	1.850	51.8	48.5	88.2	100.6
163	.472	1082	2.473	1.850	75.2	54.0	88.2	108.6
164	.472	2790	2.945	1.850	55.8	54.0	88.2	108.6

	A	B	C	D	E	F	G	H
165	.472	2298	3.103	1.850	64.7	54.0	88.2	108.6
166	.472	2790	3.395	1.850	64.3	54.0	88.2	108.6
167	.472	2778	3.552	1.850	67.4	54.0	88.2	108.6
168	.472	3146	4.474	1.850	79.8	54.0	88.2	108.6
169	.472	4621	4.676	1.850	68.8	54.0	88.2	108.6
170	.472	4904	5.980	1.850	85.4	54.0	88.2	108.6
171	.591	1610	3.687	1.969	91.9	63.3	96.8	122.7
172	.591	2753	4.294	1.969	81.8	63.3	96.8	122.7
173	.591	2962	4.317	1.969	79.3	63.3	96.8	122.7
174	.512	2950	5.598	1.969	103.1	61.4	96.8	119.9
175	.591	2962	5.800	1.969	106.6	63.3	96.8	122.7
176	.315	3466	5.531	1.969	93.9	56.5	96.8	112.9
177	.315	3466	6.115	1.969	103.9	56.5	96.8	112.9
178	.315	7461	6.857	1.969	79.4	56.5	96.8	112.9
179	.315	7461	6.924	1.969	80.2	56.5	96.8	112.9
180	.315	3810	4.654	1.969	75.4	56.5	96.8	112.9
181	.315	4302	5.980	1.969	91.2	56.5	96.8	112.9
182	.394	2851	4.946	2.008	92.6	60.6	99.7	118.9
183	.472	3073	4.294	2.087	77.5	67.1	105.6	128.4
184	.472	2126	3.327	2.087	72.2	67.1	105.6	128.4
185	.669	1389	4.294	2.087	115.2	72.3	105.6	135.8
186	.669	1770	5.103	2.087	121.3	72.3	105.6	135.8
187	.669	2704	5.980	2.087	115.0	72.3	105.6	135.8
188	.669	3577	6.924	2.087	115.8	72.3	105.6	135.8
189	.669	5592	10.027	2.087	134.1	72.3	105.6	135.8
190	.787	3073	5.958	2.165	107.5	80.3	111.7	147.2
191	.787	3073	6.857	2.165	123.7	80.3	111.7	147.2
192	.787	3073	6.610	2.165	119.2	80.3	111.7	147.2
193	.394	2520	2.833	2.165	56.4	69.6	111.7	132.1
194	.394	2569	4.047	2.165	79.8	69.6	111.7	132.1
195	.472	3208	4.609	2.165	81.4	71.8	111.7	135.1
196	.551	2765	4.991	2.165	94.9	73.9	111.7	138.1
197	.787	2999	6.632	2.165	121.1	80.3	111.7	147.2
198	.551	3220	6.700	2.165	118.1	73.9	111.7	138.1
199	.787	3245	9.487	2.165	166.6	80.3	111.7	147.2
200	.551	6809	9.532	2.165	115.5	73.9	111.7	138.1
201	.551	8641	10.904	2.165	117.3	73.9	111.7	138.1
202	.787	6231	12.882	2.165	163.2	80.3	111.7	147.2
203	.315	2851	4.317	2.165	80.8	67.5	111.7	129.1
204	.551	2790	3.912	2.244	74.1	78.8	117.8	144.9
205	.787	2360	6.228	2.244	128.2	85.5	117.8	154.2

	A	B	C	D	E	F	G	H
206	.787	5998	9.083	2.244	117.3	85.5	117.8	154.2
207	.787	7805	10.994	2.244	124.4	85.5	117.8	154.2
208	.630	1930	5.531	2.323	125.9	86.2	124.1	154.8
209	.630	7743	8.498	2.323	96.6	86.2	124.1	154.8
210	.551	3073	6.228	2.362	112.3	86.5	127.3	155.0
211	.748	8419	11.353	2.362	123.7	92.3	127.3	163.1
212	.394	3441	8.004	2.362	136.4	81.8	127.3	148.6
213	.394	3441	8.004	2.362	136.4	81.8	127.3	148.6
214	.394	6268	9.847	2.362	124.4	81.8	127.3	148.6
215	.394	6268	9.847	2.362	124.4	81.8	127.3	148.6
216	.394	6883	10.859	2.362	130.9	81.8	127.3	148.6
217	.394	3810	7.576	2.362	122.7	81.8	127.3	148.6
218	.315	2827	6.228	2.362	117.1	79.5	127.3	145.4
219	.315	3134	6.655	2.362	118.9	79.5	127.3	145.4
220	.315	3134	6.857	2.362	122.5	79.5	127.3	145.4
221	.394	6785	11.398	2.362	138.4	81.8	127.3	148.6
222	.394	2372	5.058	2.362	103.9	81.8	127.3	148.6
223	.394	2372	5.058	2.362	103.9	81.8	127.3	148.6
224	.394	4179	6.677	2.362	103.3	81.8	127.3	148.6
225	.394	3503	8.341	2.362	140.9	81.8	127.3	148.6
226	.394	3503	8.408	2.362	142.1	81.8	127.3	148.6
227	.394	3503	8.566	2.362	144.7	81.8	127.3	148.6
228	.394	3638	11.039	2.362	183.0	81.8	127.3	148.6
229	.591	2495	6.610	2.402	132.3	90.3	130.4	160.0
230	.591	7743	10.297	2.402	117.0	90.3	130.4	160.0
231	.591	2704	6.790	2.441	130.6	93.0	133.7	163.4
232	.591	6023	13.714	2.441	176.7	93.0	133.7	163.4
233	.827	3073	8.588	2.480	154.9	103.1	136.9	176.8
234	.827	1327	4.362	2.480	119.7	103.1	136.9	176.8
235	.827	1696	5.351	2.480	129.9	103.1	136.9	176.8
236	.827	2188	7.531	2.480	161.0	103.1	136.9	176.8
237	.827	2692	8.498	2.480	163.8	103.1	136.9	176.8
238	.827	3786	9.892	2.480	160.8	103.1	136.9	176.8
239	.827	5261	13.714	2.480	189.1	103.1	136.9	176.8
240	.394	1807	3.327	2.559	78.3	95.0	143.5	165.1
241	.394	3749	5.238	2.559	85.6	95.0	143.5	165.1
242	.787	2397	6.655	2.559	135.9	107.6	143.5	182.1
243	.787	2520	6.677	2.559	133.0	107.6	143.5	182.1
244	.787	2397	8.206	2.559	167.6	107.6	143.5	182.1
245	.787	3331	8.993	2.559	155.8	107.6	143.5	182.1
246	.787	6145	11.691	2.559	149.1	107.6	143.5	182.1

	A	B	C	D	E	F	G	H
247	.394	3540	8.183	2.559	137.5	95.0	143.5	165.1
248	.394	6846	8.341	2.559	100.8	95.0	143.5	165.1
249	.394	3540	8.566	2.559	144.0	95.0	143.5	165.1
250	.394	3540	8.566	2.559	144.0	95.0	143.5	165.1
251	.394	6846	8.925	2.559	107.9	95.0	143.5	165.1
252	.591	2851	6.857	2.598	128.4	104.1	146.8	177.0
253	.630	3073	6.520	2.638	117.6	108.3	150.2	182.1
254	.630	1082	4.317	2.638	131.3	108.3	150.2	182.1
255	.630	2729	7.442	2.638	142.5	108.3	150.2	182.1
256	.630	3945	10.229	2.638	162.9	108.3	150.2	182.1
257	.630	8911	13.467	2.638	142.7	108.3	150.2	182.1
258	.472	2704	7.239	2.638	139.2	103.1	150.2	175.1
259	.709	2778	6.722	2.756	127.5	120.0	160.4	195.8
260	.709	3146	7.104	2.756	126.7	120.0	160.4	195.8
261	.709	4621	9.150	2.756	134.6	120.0	160.4	195.8
262	.394	5900	7.936	2.756	103.3	109.1	160.4	181.5
263	.394	4425	7.824	2.756	117.6	109.1	160.4	181.5
264	.394	5900	8.004	2.756	104.2	109.1	160.4	181.5
265	.748	3073	8.633	2.795	155.7	124.5	163.8	201.0
266	.748	2163	7.711	2.795	165.8	124.5	163.8	201.0
267	.748	2950	9.105	2.795	167.6	124.5	163.8	201.0
268	.748	2950	9.847	2.795	181.3	124.5	163.8	201.0
269	.551	2520	5.733	2.874	114.2	123.7	170.8	198.6
270	.551	2827	7.374	2.874	138.7	123.7	170.8	198.6
271	.984	3073	9.420	2.913	169.9	142.7	174.3	222.2
272	.984	3073	11.219	2.913	202.4	142.7	174.3	222.2
273	.984	3073	14.613	2.913	263.6	142.7	174.3	222.2
274	.984	1499	7.037	2.913	181.7	142.7	174.3	222.2
275	.984	2274	8.835	2.913	185.3	142.7	174.3	222.2
276	.984	3614	13.602	2.913	226.3	142.7	174.3	222.2
277	.984	6957	23.606	2.913	283.0	142.7	174.3	222.2
278	.630	3073	9.825	2.953	177.2	132.9	177.8	208.9
279	.630	3073	10.432	2.953	188.2	132.9	177.8	208.9
280	.630	3073	10.859	2.953	195.9	132.9	177.8	208.9
281	.709	2544	9.015	2.953	178.7	135.9	177.8	212.6
282	.709	3601	12.185	2.953	203.1	135.9	177.8	212.6
283	.709	4953	14.074	2.953	200.0	135.9	177.8	212.6
284	.709	4806	15.198	2.953	219.2	135.9	177.8	212.6
285	.709	7006	17.828	2.953	213.0	135.9	177.8	212.6
286	.787	2864	8.790	3.071	164.3	148.9	188.6	226.4
287	.787	3183	10.477	3.071	185.7	148.9	188.6	226.4

	A	B	C	D	E	F	G	H
288	.787	8579	15.490	3.071	167.2	148.9	188.6	226.4
289	.472	2950	10.139	3.150	186.7	143.4	195.9	217.4
290	.984	2397	10.971	3.150	224.1	163.6	195.9	242.7
291	.472	6023	17.041	3.150	219.6	143.4	195.9	217.4
292	.472	6023	17.086	3.150	220.2	143.4	195.9	217.4
293	.394	3749	10.432	3.150	170.4	140.2	195.9	213.5
294	.394	3749	10.432	3.150	170.4	140.2	195.9	213.5
295	.472	5015	11.871	3.150	167.6	143.4	195.9	217.4
296	.472	3306	10.252	3.150	178.3	143.4	195.9	217.4
297	.472	3601	11.960	3.150	199.3	143.4	195.9	217.4
298	.472	3687	8.633	3.150	142.2	143.4	195.9	217.4
299	.472	3687	9.173	3.150	151.1	143.4	195.9	217.4
300	.472	5015	11.084	3.150	156.5	143.4	195.9	217.4
301	.472	5015	11.601	3.150	163.8	143.4	195.9	217.4
302	.472	6785	17.041	3.150	206.9	143.4	195.9	217.4
303	.709	2520	9.150	3.189	182.3	156.2	199.6	232.4
304	.787	2790	7.374	3.228	139.6	162.9	203.3	239.5
305	.630	2851	11.016	3.228	206.3	156.5	203.3	231.6
306	.984	2200	9.240	3.268	197.0	174.6	207.0	252.7
307	.984	1758	9.308	3.268	222.0	174.6	207.0	252.7
308	.984	2397	12.433	3.268	254.0	174.6	207.0	252.7
309	.984	3527	15.220	3.268	256.3	174.6	207.0	252.7
310	.984	6256	17.086	3.268	216.0	174.6	207.0	252.7
311	.984	8456	20.773	3.268	225.9	174.6	207.0	252.7
312	.866	3073	10.836	3.425	195.5	184.7	222.2	259.7
313	.866	2765	10.656	3.425	202.6	184.7	222.2	259.7
314	.866	3343	11.983	3.425	207.2	184.7	222.2	259.7
315	.866	6834	22.977	3.425	277.9	184.7	222.2	259.7
316	.945	3073	13.197	3.504	238.1	195.9	229.9	270.2
317	.945	2778	10.454	3.504	198.4	195.9	229.9	270.2
318	.945	2790	11.353	3.504	214.9	195.9	229.9	270.2
319	.945	4621	14.141	3.504	208.0	195.9	229.9	270.2
320	.866	2520	11.466	3.543	228.4	196.3	233.8	269.2
321	.866	8358	20.436	3.543	223.5	196.3	233.8	269.2
322	.984	3073	12.028	3.701	217.0	217.9	249.5	288.0
323	.945	3073	13.152	3.819	237.3	228.6	261.6	295.0
324	.945	2384	12.118	3.819	248.2	228.6	261.6	295.0
325	.945	2495	15.872	3.819	317.8	228.6	261.6	295.0
326	.945	5531	19.694	3.819	264.8	228.6	261.6	295.0
327	.945	3073	17.469	3.937	315.1	241.5	273.8	303.9
328	.945	3073	18.053	3.937	325.7	241.5	273.8	303.9

	A	B	C	D	E	F	G	H
329	.945	3073	17.761	3.937	320.4	241.5	273.8	303.9
330	.984	3011	13.647	3.937	248.7	243.5	273.8	306.1
331	.945	2249	13.961	3.937	294.4	241.5	273.8	303.9
332	.984	3024	14.726	3.937	267.8	243.5	273.8	306.1
333	.945	2446	16.232	3.937	328.2	241.5	273.8	303.9
334	.945	4953	21.718	3.937	308.6	241.5	273.8	303.9
335	.984	6907	23.134	3.937	278.3	243.5	273.8	306.1
336	.945	4806	26.349	3.937	380.1	241.5	273.8	303.9
337	.945	7583	33.116	3.937	330.3	241.5	273.8	303.9
338	.472	5039	18.772	4.134	264.4	239.3	294.6	291.1
339	1.102	1082	10.139	4.134	308.3	272.0	294.6	327.3
340	1.102	2790	12.253	4.134	232.0	272.0	294.6	327.3
341	.984	2790	12.905	4.134	244.3	265.9	294.6	320.5
342	1.102	3073	19.380	4.134	349.6	272.0	294.6	327.3
343	1.102	3970	21.515	4.134	341.5	272.0	294.6	327.3
344	1.102	7645	29.901	4.134	342.0	272.0	294.6	327.3
345	1.102	9267	35.297	4.134	366.7	272.0	294.6	327.3
346	1.102	2790	15.468	4.488	292.8	315.3	333.3	351.9
347	.630	7743	28.822	4.878	327.5	337.6	377.6	347.0
348	.630	4302	22.774	4.882	347.2	338.1	378.1	347.2
349	.630	4302	22.887	4.902	348.9	340.7	380.4	348.3
350	.630	4634	27.113	4.921	398.3	343.3	382.7	349.4
351	1.102	2249	18.997	4.921	400.6	372.5	382.7	378.9
352	1.102	2974	25.764	4.921	472.4	372.5	382.7	378.9
353	1.102	4806	38.579	4.921	556.5	372.5	382.7	378.9
354	1.102	6821	45.369	4.921	549.3	372.5	382.7	378.9
355	.630	3614	18.390	4.921	305.9	343.3	382.7	349.4
356	.630	4634	27.248	4.921	400.3	343.3	382.7	349.4
357	.630	3048	16.255	4.921	294.4	343.3	382.7	349.4
358	.630	3048	17.199	4.921	311.5	343.3	382.7	349.4
359	.630	6748	21.021	4.921	255.9	343.3	382.7	349.4
360	.630	7178	22.594	4.921	266.7	343.3	382.7	349.4
361	1.260	2249	20.324	5.827	428.5	518.9	493.0	518.9
362	1.260	3208	31.924	5.827	563.7	518.9	493.0	518.9
363	1.260	5273	45.998	5.827	633.5	518.9	493.0	518.9
364	1.260	5728	50.045	5.827	661.3	518.9	493.0	518.9
365	.787	3478	42.873	6.693	726.9	629.1	606.9	629.1
366	.787	3478	44.874	6.693	760.9	629.1	606.9	629.1
367	.945	5162	57.824	8.661	804.8	1,045.6	893.5	1,045.6
368	.945	3392	54.024	8.661	927.6	1,045.6	893.5	1,045.6
369	d7gert.pri							

	A	B	C	D	E	F	G	H
1	Anchor	Concrete	Actual	Embedment	Normalized	ACI 349-85	Kappa	VCA
2	Diamter	Strength	Load	Depth	Actual	Equation	Equation	Equation
3	in	fc	Capacity	in	Load	sqrt(lb)*in	sqrt(lb)*in	sqrt(lb)*in
4		psi	Kips		Capacity			
5					sqrt(lb)*in			
6								
7	Embedment Depth * 2 > Edge Distance							
8	.472	2827	7.419	2.953	139.5	127.1	177.8	201.4
9	.315	2827	3.597	1.969	67.7	56.5	96.8	112.9
10	.394	3134	4.721	1.693	84.3	44.4	77.2	93.2
11	Embedment Depth * 2 <= Edges Distance							
12	.350	4191	2.248	1.346	34.7	28.7	54.8	65.1
13	.315	2827	3.372	1.969	63.4	56.5	96.8	112.9
14	.315	2827	5.621	2.559	105.7	92.4	143.5	161.7
15	.472	2827	6.520	2.953	122.6	127.1	177.8	201.4
16	.472	2827	12.815	3.937	241.0	218.2	273.8	277.4
17	.630	4179	9.667	2.087	149.5	71.2	105.6	134.3
18	.787	4179	12.140	2.717	187.8	119.6	156.9	195.9
19	.984	4179	15.288	2.992	236.5	149.5	181.4	229.1
20	.630	3441	9.218	2.008	157.1	66.6	99.7	127.5
21	.236	3245	1.574	.866	27.6	12.0	28.3	30.5
22	.315	3245	2.473	.945	43.4	15.0	32.2	36.7
23	.394	3245	3.597	1.181	63.1	23.4	45.0	54.2
24	.472	3245	3.597	1.575	63.1	40.5	69.3	86.1
25	.630	3245	6.295	2.087	110.5	71.2	105.6	134.3
26	.787	3245	8.995	2.677	157.9	116.6	153.5	192.5
27	.236	3134	2.923	1.457	52.2	31.0	61.6	70.1
28	.236	3970	3.147	1.693	50.0	41.0	77.2	88.1
29	.315	3036	2.923	1.693	53.0	42.7	77.2	90.7
30	.315	3970	4.721	1.850	74.9	50.4	88.2	103.3
31	.394	3036	4.047	1.969	73.4	58.4	96.8	115.7
32	.394	3036	5.171	1.969	93.8	58.4	96.8	115.7
33	.394	3970	6.969	2.244	110.6	74.4	117.8	138.7
34	.472	3970	7.869	2.283	124.9	79.1	120.9	145.1
35	.315	3970	6.520	2.441	103.5	84.5	133.7	151.9
36	.472	2827	9.442	2.559	177.6	97.5	143.5	168.5
37	.394	3970	9.442	2.835	149.9	115.0	167.3	188.0
38	.472	3970	11.915	3.071	189.1	136.7	188.6	211.1
39	.630	2827	12.815	3.543	241.0	185.8	233.8	256.7
40	.630	2827	15.063	3.543	283.3	185.8	233.8	256.7
41	.787	3134	19.335	4.331	345.4	278.5	315.9	322.7

	A	B	C	D	E	F	G	H
42	.787	3134	19.559	4.331	349.4	278.5	315.9	322.7
43	.630	3970	20.908	4.606	331.8	303.1	346.5	331.0
44	.787	3970	25.405	5.354	403.2	413.2	434.3	413.2
45	.236	3245	2.023	.984	35.5	15.1	34.2	37.6
46	.236	3454	2.248	.984	38.3	15.1	34.2	37.6
47	.315	3245	2.473	1.181	43.4	22.2	45.0	52.3
48	.315	3454	2.923	1.181	49.7	22.2	45.0	52.3
49	.315	3245	2.923	1.181	51.3	22.2	45.0	52.3
50	.394	3245	3.597	1.575	63.1	39.0	69.3	83.8
51	.394	3454	4.272	1.575	72.7	39.0	69.3	83.8
52	.394	3663	4.272	1.575	70.6	39.0	69.3	83.8
53	.394	3245	5.171	1.693	90.8	44.4	77.2	93.2
54	.472	3245	5.396	1.969	94.7	60.4	96.8	118.5
55	.472	3454	6.295	1.969	107.1	60.4	96.8	118.5
56	.472	3663	8.993	1.969	148.6	60.4	96.8	118.5
57	.472	3245	7.194	2.087	126.3	67.1	105.6	128.4
58	.630	3245	6.520	2.362	114.5	88.8	127.3	158.2
59	.630	3663	10.567	2.362	174.6	88.8	127.3	158.2
60	.630	3454	7.869	2.520	133.9	99.7	140.2	171.9
61	.630	3245	8.993	2.559	157.9	102.6	143.5	175.3
62	.787	3245	13.040	3.150	228.9	155.8	195.9	233.0
63	.748	3454	14.838	3.189	252.5	157.8	199.6	234.3
64	.787	3245	12.140	3.268	213.1	166.5	207.0	242.8
65	.626	4388	7.621	2.500	115.1	98.2	138.5	170.0
66	.626	4388	8.183	2.500	123.5	98.2	138.5	170.0
67	.626	4388	8.183	2.500	123.5	98.2	138.5	170.0
68	.626	4388	8.183	2.500	123.5	98.2	138.5	170.0
69	.626	6379	11.151	2.500	139.6	98.2	138.5	170.0
70	.626	6379	12.118	2.500	151.7	98.2	138.5	170.0
71	.626	6379	12.680	2.500	158.8	98.2	138.5	170.0
72	.500	3491	6.700	2.500	113.4	94.2	138.5	164.7
73	.500	3491	6.812	2.500	115.3	94.2	138.5	164.7
74	.500	3491	5.013	2.500	84.9	94.2	138.5	164.7
75	.500	3491	6.655	2.500	112.6	94.2	138.5	164.7
76	.500	4548	7.352	2.500	109.0	94.2	138.5	164.7
77	.500	4744	8.071	2.500	117.2	94.2	138.5	164.7
78	.500	4744	8.318	2.500	120.8	94.2	138.5	164.7
79	.500	4744	8.318	2.500	120.8	94.2	138.5	164.7
80	.626	1967	7.104	2.819	160.2	122.0	165.9	197.4
81	.472	5789	13.264	3.150	174.3	143.4	195.9	217.4
82	.472	4339	13.939	3.150	211.6	143.4	195.9	217.4

	A	B	C	D	E	F	G	H
83	.472	5789	12.298	3.150	161.6	143.4	195.9	217.4
84	.768	3503	14.906	3.252	251.8	164.3	205.5	240.5
85	.768	3700	12.905	3.252	212.2	164.3	205.5	240.5
86	.768	3700	15.603	3.252	256.5	164.3	205.5	240.5
87	.768	5642	14.164	3.268	188.6	165.7	207.0	241.8
88	.768	1967	8.386	3.441	189.1	182.0	223.7	255.9
89	.787	3331	35.364	6.693	612.8	629.1	606.9	629.1
90	d7eng.pri							

	A	B	C	D	E	F	G	H
1	Anchor	Concrete	Actual	Embedment	Normalized	ACI 349-85	Kappa	VCA
2	Diameter	Strength	Load	Depth	Actual	Equation	Equation	Equation
3	in	fc	Capacity	in	Load	sqrt(lb)*in	sqrt(lb)*in	sqrt(lb)*in
4		psi	Kips		Capacity			
5					sqrt(lb)*in			
6								
7	Embedment Depth * 2 > Edge Distance							
8	.236	3503	2.406	1.091	40.64	18.18	39.92	44.40
9	.350	3503	3.507	1.311	59.26	27.37	52.61	62.47
10	.350	2286	2.001	1.346	41.85	28.71	54.76	65.08
11	.350	4191	2.249	1.346	34.73	28.71	54.76	65.08
12	.374	2114	3.103	1.575	67.48	38.57	69.27	83.17
13	.374	2114	3.417	1.575	74.32	38.57	69.27	83.17
14	.500	3196	5.351	2.031	94.65	64.63	101.49	124.80
15	.500	3097	6.295	2.031	113.11	64.63	101.49	124.80
16	.500	5002	7.352	2.031	103.94	64.63	101.49	124.80
17	.500	3097	7.397	2.031	132.90	64.63	101.49	124.80
18	.500	3503	7.801	2.031	131.81	64.63	101.49	124.80
19	.500	3097	8.228	2.031	147.85	64.63	101.49	124.80
20	.500	5002	9.195	2.031	130.01	64.63	101.49	124.80
21	.500	6195	10.094	2.031	128.26	64.63	101.49	124.80
22	.500	6195	10.409	2.031	132.25	64.63	101.49	124.80
23	.500	5002	10.881	2.031	153.85	64.63	101.49	124.80
24	.500	6195	11.196	2.031	142.25	64.63	101.49	124.80
25	.500	3196	6.790	2.031	120.11	64.63	101.49	124.80
26	.626	3503	10.499	2.469	177.39	95.99	135.94	167.29
27	.874	3503	17.806	3.689	300.85	211.53	248.34	281.10
28	Embedment Depth * 2 <= Edge Distance							
29	.236	3970	1.754	1.000	27.83	15.53	35.05	38.59
30	.236	3970	1.956	1.000	31.04	15.53	35.05	38.59
31	.236	3970	2.091	1.000	33.18	15.53	35.05	38.59
32	.374	2028	2.046	1.024	45.43	17.98	36.30	42.97
33	.236	5642	2.855	1.102	38.01	18.54	40.57	45.17
34	.236	2028	1.484	1.138	32.95	19.65	42.54	47.52
35	.236	3970	1.911	1.252	30.33	23.41	49.10	55.34
36	.236	3970	2.361	1.252	37.47	23.41	49.10	55.34
37	.374	1979	2.451	1.398	55.09	31.12	57.91	69.53
38	.374	4240	3.058	1.488	46.95	34.83	63.63	76.44
39	.394	3503	5.103	1.531	86.23	37.05	66.43	80.37
40	.374	2286	3.237	1.531	67.71	36.67	66.43	79.79
41	.394	5642	3.754	1.535	49.99	37.22	66.69	80.68

	A	B	C	D	E	F	G	H
42	.374	4240	3.170	1.547	48.68	37.36	67.46	81.02
43	.350	3909	1.551	1.575	24.81	38.10	69.27	82.46
44	.350	3909	1.776	1.575	28.41	38.10	69.27	82.46
45	.350	3909	1.799	1.575	28.77	38.10	69.27	82.46
46	.350	3909	1.911	1.575	30.57	38.10	69.27	82.46
47	.350	3909	2.203	1.575	35.24	38.10	69.27	82.46
48	.236	3736	3.372	1.626	55.17	38.05	72.67	82.96
49	.500	2212	4.721	2.000	100.38	62.83	99.14	122.14
50	.500	2212	5.396	2.000	114.71	62.83	99.14	122.14
51	.500	3196	7.127	2.031	126.07	64.63	101.49	124.80
52	.500	3196	6.542	2.031	115.73	64.63	101.49	124.80
53	.500	3196	6.272	2.031	110.96	64.63	101.49	124.80
54	.500	3196	7.307	2.031	129.25	64.63	101.49	124.80
55	.500	5642	7.869	2.047	104.76	65.53	102.67	126.13
56	.350	4179	6.902	2.244	106.77	73.16	117.83	136.98
57	.626	3884	7.509	2.244	120.49	80.94	117.83	147.82
58	.500	4007	7.329	2.264	115.79	78.62	119.38	144.54
59	.472	2102	4.789	2.291	104.45	79.58	121.57	145.79
60	.472	3626	5.216	2.319	86.62	81.34	123.77	148.13
61	.472	1881	4.272	2.370	98.50	84.66	127.89	152.48
62	.472	4007	5.261	2.390	83.11	85.95	129.49	154.15
63	.472	1979	4.272	2.394	96.02	86.21	129.81	154.48
64	.626	4388	7.621	2.500	115.06	98.21	138.55	170.02
65	.626	4388	8.183	2.500	123.54	98.21	138.55	170.02
66	.626	4388	8.183	2.500	123.54	98.21	138.55	170.02
67	.626	4388	8.183	2.500	123.54	98.21	138.55	170.02
68	.626	6379	11.151	2.500	139.62	98.21	138.55	170.02
69	.626	6379	12.118	2.500	151.72	98.21	138.55	170.02
70	.626	6379	12.680	2.500	158.76	98.21	138.55	170.02
71	.500	3491	6.700	2.500	113.40	94.25	138.55	164.67
72	.500	3491	6.812	2.500	115.30	94.25	138.55	164.67
73	.500	3491	5.013	2.500	84.86	94.25	138.55	164.67
74	.500	3491	6.655	2.500	112.64	94.25	138.55	164.67
75	.500	4548	7.352	2.500	109.02	94.25	138.55	164.67
76	.500	4744	8.071	2.500	117.18	94.25	138.55	164.67
77	.500	4744	8.318	2.500	120.77	94.25	138.55	164.67
78	.500	4744	8.318	2.500	120.77	94.25	138.55	164.67
79	.626	1967	7.104	2.819	160.20	122.03	165.89	197.37
80	.472	5789	13.264	3.150	174.34	143.36	195.92	217.43
81	.472	4339	13.939	3.150	211.62	143.36	195.92	217.43
82	.472	5789	12.298	3.150	161.63	143.36	195.92	217.43

	A	B	C	D	E	F	G	H
83	.768	3503	14.906	3.252	251.85	164.27	205.55	240.50
84	.768	3700	12.905	3.252	212.16	164.27	205.55	240.50
85	.768	3700	15.603	3.252	256.52	164.27	205.55	240.50
86	.768	5642	14.164	3.268	188.57	165.71	207.04	241.80
87	.768	1967	8.386	3.441	189.10	181.98	223.72	255.86
88	.787	3331	35.364	6.693	612.76	629.14	606.89	629.14
89	d6usa.pri							

	A	B	C	D	E	F	G	H
1	Anchor	Concrete	Actual	Embedment	Normalized	ACI 349-85	Kappa	VCA
2	Diameter	Strength	Load	Depth	Actual	Equation	Equation	Equation
3	in	fc	Capacity	in	Load	sqrt(lb)*in	sqrt(lb)*in	sqrt(lb)*in
4		psi	Kips		Capacity			
5					sqrt(lb)*in			
6	Embedment Depth * 2 <= Edge Distance							
7	.315	2225	3.103	1.654	65.8	40.9	74.5	87.5
8	.315	2237	4.631	2.756	97.9	106.3	160.4	177.9
9	.315	2311	4.204	2.087	87.5	63.0	105.6	122.6
10	.315	2311	4.362	2.283	90.7	74.6	120.9	138.9
11	.315	2606	4.429	2.402	86.8	82.0	130.4	148.7
12	.315	2704	4.879	2.402	93.8	82.0	130.4	148.7
13	.315	3257	5.576	2.500	97.7	88.4	138.5	156.8
14	.315	3257	5.576	2.500	97.7	88.4	138.5	156.8
15	.315	3257	6.025	2.500	105.6	88.4	138.5	156.8
16	.315	3257	6.025	2.500	105.6	88.4	138.5	156.8
17	.315	3429	5.710	2.461	97.5	85.8	135.3	153.5
18	.315	3896	7.531	2.402	120.7	82.0	130.4	148.7
19	.315	4302	5.171	2.087	78.8	63.0	105.6	122.6
20	.315	4302	5.890	2.283	89.8	74.6	120.9	138.9
21	.315	4302	6.452	2.480	98.4	87.1	136.9	155.2
22	.315	4560	4.564	1.654	67.6	40.9	74.5	87.5
23	.394	3036	6.857	2.756	124.4	109.1	160.4	181.5
24	.394	3036	7.329	2.756	133.0	109.1	160.4	181.5
25	.394	3257	7.531	2.874	132.0	118.0	170.8	191.2
26	.394	3257	8.588	2.874	150.5	118.0	170.8	191.2
27	.394	3429	8.701	2.795	148.6	112.0	163.8	184.7
28	.394	3429	8.746	2.795	149.3	112.0	163.8	184.7
29	.394	4228	10.769	2.874	165.6	118.0	170.8	191.2
30	.394	4474	11.151	2.874	166.7	118.0	170.8	191.2
31	.472	1856	6.385	3.012	148.2	131.9	183.2	206.2
32	.472	1856	5.980	3.031	138.8	133.5	185.0	207.8
33	.472	1856	6.362	3.031	147.7	133.5	185.0	207.8
34	.472	1856	7.374	3.031	171.2	133.5	185.0	207.8
35	.472	2212	5.553	2.362	118.1	84.1	127.3	151.8
36	.472	2225	9.915	3.976	210.2	222.3	277.9	280.2
37	.472	2311	7.891	3.031	164.2	133.5	185.0	207.8
38	.472	2311	8.746	3.268	181.9	153.6	207.0	226.9
39	.472	2434	9.038	3.071	183.2	136.7	188.6	211.1
40	.472	2434	9.240	3.071	187.3	136.7	188.6	211.1
41	.472	2913	11.151	3.012	206.6	131.9	183.2	206.2

	A	B	C	D	E	F	G	H
42	.472	2913	11.871	3.071	219.9	136.7	188.6	211.1
43	.472	3564	13.062	3.071	218.8	136.7	188.6	211.1
44	.472	4302	14.996	3.031	228.6	133.5	185.0	207.8
45	.472	4302	16.165	3.228	246.5	150.1	203.3	223.7
46	.472	4302	16.817	3.425	256.4	167.8	222.2	239.3
47	.472	4916	16.547	3.937	236.0	218.2	273.8	277.4
48	.472	5162	10.432	2.362	145.2	84.1	127.3	151.8
49	.472	5162	10.679	2.362	148.6	84.1	127.3	151.8
50	.472	5371	13.961	3.031	190.5	133.5	185.0	207.8
51	.472	6109	15.130	3.012	193.6	131.9	183.2	206.2
52	.630	5015	20.638	3.976	291.4	230.2	277.9	289.0
53	.630	5629	20.841	3.976	277.8	230.2	277.9	289.0
54	.630	3491	18.525	3.976	313.6	230.2	277.9	289.0
55	.630	3368	16.727	3.976	298.2	230.2	277.9	289.0
56	.630	3491	17.536	3.976	296.8	230.2	277.9	289.0
57	.630	4056	18.278	3.976	287.0	230.2	277.9	289.0
58	.630	3527	14.096	3.976	237.3	230.2	277.9	289.0
59	.787	2053	18.300	4.173	403.9	260.1	298.8	311.9
60	.787	5162	26.978	4.173	375.5	260.1	298.8	311.9
61	.787	4265	29.069	4.724	445.1	327.2	359.9	347.7
62	.787	6723	33.656	4.724	410.5	327.2	359.9	347.7
63	.787	6743	35.162	4.724	428.0	327.2	359.9	347.7
64	.787	1672	14.906	4.921	364.6	353.0	382.7	359.2
65	.787	3368	29.272	4.921	504.4	353.0	382.7	359.2
66	.787	4720	36.691	4.921	534.1	353.0	382.7	359.2
67	.787	2335	18.031	4.961	373.1	358.3	387.3	361.4
68	.787	2274	22.190	4.961	465.3	358.3	387.3	361.4
69	.787	3466	26.506	4.961	450.2	358.3	387.3	361.4
70	.787	4093	27.001	4.961	422.1	358.3	387.3	361.4
71	.787	3466	29.496	4.961	501.0	358.3	387.3	361.4
72	.787	3429	22.707	4.961	387.8	358.3	387.3	361.4
73	.787	3995	23.494	4.961	371.7	358.3	387.3	361.4
74	.787	3233	24.798	4.961	436.2	358.3	387.3	361.4
75	.787	3060	25.809	5.354	466.5	413.2	434.3	413.2
76	.787	4720	31.835	5.394	463.4	419.0	439.1	419.0
77	.787	1942	21.853	5.748	495.9	472.1	483.0	472.1
78	.787	4437	34.442	5.787	517.1	478.2	488.0	478.2
79	.787	2053	26.124	6.142	576.6	534.8	533.5	534.8
80	.787	5162	42.536	6.142	592.0	534.8	533.5	534.8
81	.315	3429	4.811	2.362	82.2	79.5	127.3	145.4
82	.315	3896	6.812	2.362	109.1	79.5	127.3	145.4

	A	B	C	D	E	F	G	H
83	.394	3036	6.497	2.717	117.9	106.2	156.9	178.2
84	.472	2434	7.531	3.012	152.7	131.9	183.2	206.2
85	.472	6109	14.119	3.012	180.6	131.9	183.2	206.2
86	d7fra.pri							

	A	B	C	D	E	F	G	H
1–5	Anchor Diameter in	Concrete Strength fc psi	Actual Load Capacity Kips	Embedment Depth in	Normalized Actual Load Capacity sqrt(lb)*in	ACI 349-85 Equation sqrt(lb)*in	Kappa Equation sqrt(lb)*in	VCA Equation sqrt(lb)*in
6								
7	Embedment Depth * 2 <= Edge Distance							
8	.394	2851	3.192	1.260	59.8	26.2	49.6	59.8
9	.394	3282	2.833	1.260	49.4	26.2	49.6	59.8
10	.315	4978	3.799	1.378	53.9	29.3	56.7	66.5
11	.315	3196	3.170	1.417	56.1	30.9	59.1	69.4
12	.394	2851	3.620	1.417	67.8	32.3	59.1	71.6
13	.315	4990	3.732	1.457	52.8	32.4	61.6	72.4
14	.315	3159	3.485	1.496	62.0	34.0	64.1	75.3
15	.394	4990	4.384	1.535	62.1	37.2	66.7	80.7
16	.394	3196	3.327	1.614	58.9	40.7	71.9	86.9
17	.394	4978	4.789	1.614	67.9	40.7	71.9	86.9
18	.236	2839	3.080	1.693	57.8	41.0	77.2	88.1
19	.236	3171	3.372	1.732	59.9	42.9	79.9	91.2
20	.394	2237	3.957	1.811	83.7	50.2	85.4	102.7
21	.630	3060	6.025	1.890	108.9	59.8	91.1	117.4
22	.630	3159	6.587	2.008	117.2	66.6	99.7	127.5
23	.394	2876	4.024	2.165	75.0	69.6	111.7	132.1
24	.472	5236	6.969	2.244	96.3	76.6	117.8	141.8
25	.630	3159	7.509	2.283	133.6	83.6	120.9	151.4
26	.315	3306	6.003	2.362	104.4	79.5	127.3	145.4
27	.394	5236	7.869	2.480	108.7	89.6	136.9	158.5
28	.394	2839	5.328	2.520	100.0	92.2	140.2	161.8
29	.630	2544	6.272	2.559	124.4	102.6	143.5	175.3
30	.630	4978	10.589	2.638	150.1	108.3	150.2	182.1
31	.472	3306	8.993	2.835	156.4	117.8	167.3	191.6
32	.472	5371	10.409	2.835	142.0	117.8	167.3	191.6
33	.630	3552	11.151	4.055	187.1	238.7	286.2	294.6
34	.630	5261	15.513	4.094	213.9	243.1	290.4	297.4
35	.630	2003	17.131	4.921	382.7	343.3	382.7	349.4
36	d7swe.pri							

	A	B	C	D	E	F	G	H
1–5	Anchor Diameter in	Concrete Strength fc psi	Actual Load Capacity Kips	Embedment Depth in	Normalized Actual Load Capacity sqrt(lb)*in	ACI 349-85 Equation sqrt(lb)*in	Kappa Equation sqrt(lb)*in	VCA Equation sqrt(lb)*in
6								
7	Embedment Depth * 2 <= Edges Distance							
8	1.063	3798	40.468	5.512	656.7	455.4	520.7	455.4
9	1.063	3798	42.716	5.591	693.1	467.4	531.9	467.4
10	1.181	6969	46.313	5.118	554.8	405.1	465.9	405.1
11	k4swe.pri							

APPENDIX D: SPREADSHEET DATA BASE FOR CONCRETE FAILURE (SI UNITS)

	A	B	C	D	E	F	G	H
1	Bolt	Concrete	Actual	Embedment	Normalized	ACI 349-85	KAPPA	VCA
2	Diameter	Strength	Load	Depth	Actual	Equation	Equation	Equation
3	mm	fcc	Capacity	mm	Load	sqrt(N)*mm	sqrt(N)*mm	sqrt(N)*mm
4		N/mm^2	KN		Capacity			
5					sqrt(N)*mm			
6								
7	Embedment Depth * 2 > Edge Distance							
8	19.5	43.1	133.8	193.6	20381	39606	41753	39606
9	19.5	43.1	140.1	193.6	21340	39606	41753	39606
10	19.5	39.6	88.5	88.9	14064	9251	12992	12843
11	22.2	39.8	191.2	193.6	30307	40108	41753	40108
12	25.4	35.1	435.9	251.4	73576	66804	61785	66804
13	Embedment Depth * 2 <= Edge Distance							
14	19.5	24.4	53.3	92.1	10790	9867	13700	13330
15	19.5	42.1	82.2	92.1	12669	9867	13700	13330
16	19.5	25.3	71.1	63.5	14135	5060	7843	8676
17	19.5	24.4	48.9	92.1	9900	9867	13700	13330
18	19.5	42.1	82.2	92.1	12669	9867	13700	13330
19	19.5	25.3	112.9	88.9	22446	9251	12992	12843
20	19.5	24.4	62.2	92.1	12592	9867	13700	13330
21	19.5	42.1	76.9	92.1	11852	9867	13700	13330
22	25.4	27.1	62.3	76.0	11968	7398	10270	11329
23	25.4	25.3	100.1	127.0	19901	18581	22184	18581
24	25.4	25.2	117.4	127.0	23387	18581	22184	18581
25	12.7	24.3	34.7	93.7	7039	9571	14059	12855
26	15.9	24.3	62.3	87.3	12638	8649	12643	12231
27	15.9	24.3	55.2	87.3	11198	8649	12643	12231
28	15.9	24.3	41.8	87.3	8480	8649	12643	12231
29	19.0	24.3	62.3	87.3	12638	8909	12643	12544
30	19.0	24.3	48.0	88.9	9737	9209	12992	12792
31	19.0	24.3	48.0	88.9	9737	9209	12992	12792
32	19.0	24.3	55.2	88.9	11198	9209	12992	12792
33	19.0	24.3	52.0	88.9	10549	9209	12992	12792
34	22.2	24.3	48.0	88.9	9737	9482	12992	13118
35	22.2	24.3	55.2	88.9	11198	9482	12992	13118
36	22.2	24.3	62.3	88.9	12638	9482	12992	13118
37	k3usa.pri							

	A	B	C	D	E	F	G	H
1	Bolt	Concrete	Actual	Embedment	Normalized	ACI 349-85	KAPPA	VCA
2	Diameter	Strength	Load	Depth	Actual	Equation	Equation	Equation
3	mm	fcc	Capacity	mm	Load	sqrt(N)*mm	sqrt(N)*mm	sqrt(N)*mm
4		N/mm^2	KN		Capacity			
5					sqrt(N)*mm			
6								
7	Embedment Depth * 2 > Edge Distance							
8	9.5	13.8	13.2	42.9	3553	2158	4355	4545
9	9.5	13.8	14.0	42.9	3769	2158	4355	4545
10	9.5	13.8	14.8	42.9	3984	2158	4355	4545
11	9.5	28.4	20.2	42.9	3790	2158	4355	4545
12	9.5	28.4	21.2	42.9	3978	2158	4355	4545
13	9.5	28.4	21.2	42.9	3978	2158	4355	4545
14	15.9	13.8	32.0	67.1	8614	5347	8520	8982
15	15.9	13.8	32.8	67.1	8829	5347	8520	8982
16	15.9	13.8	33.2	67.1	8937	5347	8520	8982
17	15.9	28.4	47.6	67.1	8932	5347	8520	8982
18	15.9	28.4	49.6	67.1	9307	5347	8520	8982
19	15.9	28.4	52.8	67.1	9908	5347	8520	8982
20	22.2	13.8	46.2	90.3	12437	9752	13300	13335
21	22.2	13.8	46.4	90.3	12490	9752	13300	13335
22	22.2	13.8	47.8	90.3	12867	9752	13300	13335
23	22.2	28.4	66.8	90.3	12535	9752	13300	13335
24	22.2	28.4	70.4	90.3	13210	9752	13300	13335
25	22.2	28.4	71.2	90.3	13360	9752	13300	13335
26	50.0	27.7	868.3	525.0	164980	289800	186454	289800
27	50.0	19.1	731.3	525.0	167332	289800	186454	289800
28	50.0	19.1	770.1	525.0	176210	289800	186454	289800
29	50.0	19.1	841.9	525.0	192639	289800	186454	289800
30	50.0	23.7	843.6	525.0	173286	289800	186454	289800
31	50.0	23.7	860.3	525.0	176716	289800	186454	289800
32	50.0	19.1	861.7	525.0	197169	289800	186454	289800
33	50.0	23.7	884.0	525.0	181584	289800	186454	289800
34	50.0	23.7	917.6	525.0	188486	289800	186454	289800
35	50.0	27.7	885.2	525.0	168191	289800	186454	289800
36	50.0	19.1	838.6	525.0	191884	289800	186454	289800
37	50.0	23.7	852.4	525.0	175093	289800	186454	289800
38	50.0	23.7	919.3	525.0	188835	289800	186454	289800
39	50.0	19.1	767.2	525.0	175546	289800	186454	289800
40	Embedment Depth * 2 <= Edge Distance							
41	9.5	27.9	15.2	42.9	2878	2158	4355	4545

	A	B	C	D	E	F	G	H
42	9.5	28.4	16.0	42.9	3002	2158	4355	4545
43	9.5	25.1	18.2	42.9	3633	2158	4355	4545
44	9.5	25.1	19.2	42.9	3832	2158	4355	4545
45	9.5	27.9	20.6	42.9	3900	2158	4355	4545
46	9.5	27.9	20.6	42.9	3900	2158	4355	4545
47	9.5	25.1	20.8	42.9	4152	2158	4355	4545
48	9.5	25.1	20.8	42.9	4152	2158	4355	4545
49	9.5	27.9	21.0	42.9	3976	2158	4355	4545
50	9.5	27.9	21.2	42.9	4014	2158	4355	4545
51	9.5	25.1	22.0	42.9	4391	2158	4355	4545
52	9.5	27.9	22.6	42.9	4279	2158	4355	4545
53	9.5	35.2	23.0	42.9	3877	2158	4355	4545
54	9.5	28.4	24.0	42.9	4504	2158	4355	4545
55	9.5	25.1	24.8	42.9	4950	2158	4355	4545
56	9.5	36.5	25.0	42.9	4138	2158	4355	4545
57	9.5	36.5	25.0	42.9	4138	2158	4355	4545
58	9.5	35.2	25.5	42.9	4298	2158	4355	4545
59	9.5	36.5	27.0	42.9	4469	2158	4355	4545
60	9.5	35.2	27.0	42.9	4551	2158	4355	4545
61	9.5	35.2	27.0	42.9	4551	2158	4355	4545
62	9.5	35.2	27.5	42.9	4635	2158	4355	4545
63	9.5	35.2	29.5	42.9	4972	2158	4355	4545
64	22.2	30.5	30.5	65.3	5523	5485	8179	9210
65	22.2	30.5	41.4	65.3	7496	5485	8179	9210
66	22.2	30.5	42.4	65.3	7677	5485	8179	9210
67	15.9	22.7	37.0	67.1	7766	5347	8520	8982
68	15.9	22.7	40.5	67.1	8500	5347	8520	8982
69	15.9	22.7	42.0	67.1	8815	5347	8520	8982
70	15.9	22.7	43.5	67.1	9130	5347	8520	8982
71	15.9	22.7	44.0	67.1	9235	5347	8520	8982
72	15.9	22.7	49.5	67.1	10389	5347	8520	8982
73	15.9	11.4	29.0	67.1	8589	5347	8520	8982
74	15.9	11.4	32.0	67.1	9478	5347	8520	8982
75	15.9	11.4	37.0	67.1	10958	5347	8520	8982
76	15.9	22.7	37.5	67.1	7871	5347	8520	8982
77	15.9	22.7	39.0	67.1	8186	5347	8520	8982
78	15.9	22.7	40.0	67.1	8396	5347	8520	8982
79	15.9	22.7	40.0	67.1	8396	5347	8520	8982
80	15.9	22.7	42.0	67.1	8815	5347	8520	8982
81	15.9	22.7	45.0	67.1	9445	5347	8520	8982
82	15.9	46.4	54.0	67.1	7927	5347	8520	8982

	A	B	C	D	E	F	G	H
83	15.9	46.4	56.0	67.1	8221	5347	8520	8982
84	15.9	46.4	57.0	67.1	8368	5347	8520	8982
85	15.9	37.3	57.5	67.1	9415	5347	8520	8982
86	15.9	37.3	58.0	67.1	9497	5347	8520	8982
87	15.9	37.3	59.0	67.1	9660	5347	8520	8982
88	15.9	46.4	59.0	67.1	8661	5347	8520	8982
89	15.9	46.4	60.0	67.1	8808	5347	8520	8982
90	15.9	46.4	61.0	67.1	8955	5347	8520	8982
91	22.2	30.5	69.2	90.3	12530	9752	13300	13335
92	22.2	30.5	71.2	90.3	12892	9752	13300	13335
93	22.2	30.5	76.0	90.3	13761	9752	13300	13335
94	22.2	46.4	78.5	90.3	11524	9752	13300	13335
95	22.2	46.4	80.0	90.3	11744	9752	13300	13335
96	22.2	46.4	81.5	90.3	11965	9752	13300	13335
97	22.2	46.4	82.0	90.3	12038	9752	13300	13335
98	22.2	46.4	83.5	90.3	12258	9752	13300	13335
99	22.2	46.4	92.0	90.3	13506	9752	13300	13335
100	22.2	27.9	62.0	90.3	11738	9752	13300	13335
101	22.2	27.9	64.0	90.3	12117	9752	13300	13335
102	22.2	27.9	64.8	90.3	12268	9752	13300	13335
103	22.2	27.9	65.6	90.3	12419	9752	13300	13335
104	22.2	25.1	67.6	90.3	13493	9752	13300	13335
105	22.2	27.9	68.4	90.3	12950	9752	13300	13335
106	22.2	27.9	68.4	90.3	12950	9752	13300	13335
107	22.2	28.4	69.0	90.3	12948	9752	13300	13335
108	22.2	35.2	73.0	90.3	12304	9752	13300	13335
109	22.2	25.1	73.2	90.3	14611	9752	13300	13335
110	22.2	28.4	74.0	90.3	13886	9752	13300	13335
111	22.2	35.2	74.0	90.3	12473	9752	13300	13335
112	22.2	35.2	74.0	90.3	12473	9752	13300	13335
113	22.2	25.1	75.6	90.3	15090	9752	13300	13335
114	22.2	36.5	76.0	90.3	12580	9752	13300	13335
115	22.2	28.4	76.0	90.3	14261	9752	13300	13335
116	22.2	25.1	76.0	90.3	15170	9752	13300	13335
117	22.2	30.5	76.8	90.3	13906	9752	13300	13335
118	22.2	30.5	77.6	90.3	14051	9752	13300	13335
119	22.2	35.2	78.0	90.3	13147	9752	13300	13335
120	22.2	25.1	78.8	90.3	15729	9752	13300	13335
121	22.2	25.1	79.2	90.3	15808	9752	13300	13335
122	22.2	35.2	80.0	90.3	13484	9752	13300	13335
123	22.2	30.5	80.0	90.3	14486	9752	13300	13335

	A	B	C	D	E	F	G	H
124	22.2	36.5	82.0	90.3	13573	9752	13300	13335
125	22.2	36.5	82.5	90.3	13656	9752	13300	13335
126	22.2	35.2	85.0	90.3	14327	9752	13300	13335
127	15.9	15.5	43.2	92.1	10973	9549	13700	12956
128	15.9	15.5	47.6	92.1	12090	9549	13700	12956
129	15.9	15.5	50.0	92.1	12700	9549	13700	12956
130	15.9	11.4	42.0	92.1	12439	9549	13700	12956
131	15.9	11.4	46.0	92.1	13624	9549	13700	12956
132	15.9	11.4	51.0	92.1	15105	9549	13700	12956
133	15.9	37.3	88.0	92.1	14409	9549	13700	12956
134	15.9	37.3	91.0	92.1	14900	9549	13700	12956
135	15.9	37.3	98.0	92.1	16046	9549	13700	12956
136	19.0	20.3	72.0	115.3	15980	14865	19190	16459
137	19.0	20.3	72.0	115.3	15980	14865	19190	16459
138	19.0	20.3	87.2	115.3	19354	14865	19190	16459
139	22.2	30.5	94.4	115.3	17093	15220	19190	16833
140	22.2	30.5	97.6	115.3	17673	15220	19190	16833
141	22.2	30.5	100.8	115.3	18252	15220	19190	16833
142	22.0	27.7	97.2	130.0	18468	18970	22975	18970
143	22.2	18.8	85.6	140.3	19742	21887	25758	21887
144	22.0	24.8	173.5	185.0	34840	36763	39002	36763
145	22.0	22.2	150.6	185.0	31963	36763	39002	36763
146	22.0	22.2	151.7	185.0	32197	36763	39002	36763
147	22.0	22.2	145.6	185.0	30902	36763	39002	36763
148	22.0	27.1	187.0	185.0	35922	36763	39002	36763
149	22.0	27.1	200.0	185.0	38419	36763	39002	36763
150	22.0	28.2	207.0	185.0	38980	36763	39002	36763
151	22.0	29.7	223.0	185.0	40919	36763	39002	36763
152	22.0	29.6	223.0	185.0	40988	36763	39002	36763
153	22.0	33.1	223.0	185.0	38761	36763	39002	36763
154	22.0	27.1	226.0	185.0	43413	36763	39002	36763
155	22.0	29.6	228.0	185.0	41907	36763	39002	36763
156	22.0	27.8	230.0	185.0	43622	36763	39002	36763
157	22.0	27.1	236.0	185.0	45334	36763	39002	36763
158	22.0	29.7	239.0	185.0	43855	36763	39002	36763
159	22.0	30.0	241.0	185.0	44000	36763	39002	36763
160	22.0	29.6	259.0	185.0	47605	36763	39002	36763
161	22.0	22.2	148.4	185.0	31496	36763	39002	36763
162	22.0	15.6	128.0	185.0	32408	36763	39002	36763
163	22.0	16.4	128.4	185.0	31706	36763	39002	36763
164	22.0	16.4	135.0	185.0	33336	36763	39002	36763

	A	B	C	D	E	F	G	H
165	22.0	15.6	138.6	185.0	35091	36763	39002	36763
166	22.0	20.2	181.5	185.0	40383	36763	39002	36763
167	22.0	20.2	182.3	185.0	40561	36763	39002	36763
168	22.0	19.9	182.7	185.0	40955	36763	39002	36763
169	22.0	19.9	201.0	185.0	45058	36763	39002	36763
170	22.0	22.2	141.3	185.0	29989	36763	39002	36763
171	30.0	44.0	411.8	250.0	62081	67200	61269	67200
172	30.0	34.0	412.1	250.0	70675	67200	61269	67200
173	30.0	44.0	415.1	250.0	62579	67200	61269	67200
174	30.0	44.0	435.6	250.0	65669	67200	61269	67200
175	30.0	34.0	436.4	250.0	74842	67200	61269	67200
176	30.0	70.9	499.0	250.0	59262	67200	61269	67200
177	30.0	70.4	535.0	250.0	63763	67200	61259	67200
178	30.0	70.9	559.1	250.0	66400	67200	61269	67200
179	30.0	71.9	617.5	250.0	72824	67200	61269	67200
180	30.0	71.9	620.5	250.0	73177	67200	61269	67200
181	30.0	71.9	624.2	250.0	73614	67200	61269	67200
182	30.0	22.2	294.3	260.0	62462	72384	64982	72384
183	30.0	22.2	302.1	260.0	64117	72384	64982	72384
184	30.0	22.2	262.3	260.0	55670	72384	64982	72384
185	30.0	22.2	266.6	260.0	56583	72384	64982	72384
186	30.0	22.2	270.7	260.0	57453	72384	64982	72384
187	30.0	22.2	273.6	260.0	58068	72384	64982	72384
188	30.0	22.2	278.0	260.0	59002	72384	64982	72384
189	30.0	22.2	285.9	260.0	60679	72384	64982	72384
190	30.0	27.7	290.0	260.0	55101	72384	64982	72384
191	40.0	22.2	450.6	355.0	95635	134616	103675	134616
192	40.0	22.2	452.0	355.0	95932	134616	103675	134616
193	40.0	22.2	465.7	355.0	98839	134616	103675	134616
194	40.0	22.2	468.0	355.0	99327	134616	103675	134616
195	40.0	22.2	479.7	355.0	101811	134616	103675	134616
196	40.0	22.2	470.5	355.0	99858	134616	103675	134616
197	40.0	22.2	475.6	355.0	100941	134616	103675	134616
198	40.0	22.2	421.6	355.0	89480	134616	103675	134616
199	40.0	22.2	395.0	355.0	83934	134616	103675	134616
200	40.0	22.2	439.3	355.0	93236	134616	103675	134616
201	k4ger.pri							

	A	B	C	D	E	F	G	H
1	Bolt	Concrete	Load	Embedment	Normalized	ACI 349-85	VCA	KAPPA
2	Diameter	Strength	Capacity	Depth	Actual	Equation	Equation	Equation
3	mm	fcc	KN	mm	Load	sqrt(N)*mm	sqrt(N)*mm	sqrt(N)*mm
4		N/mm^2			Capacity			
5					sqrt(N)*mm			
6								
7	Embedment Depth * 2 > Edge Distance							
8	6.0	32.3	9.1	28.0	1601	914	2000	2226
9	12.0	25.0	12.9	31.0	2580	1280	2330	2904
10	12.0	25.0	13.8	37.0	2760	1740	3038	3785
11	6.0	24.0	14.3	37.0	2919	1527	3038	3456
12	8.0	23.0	15.0	40.0	3128	1843	3415	4012
13	8.0	59.0	26.2	40.0	3411	1843	3415	4012
14	8.0	26.0	16.0	40.0	3138	1843	3415	4012
15	8.0	29.8	18.1	40.0	3316	1843	3415	4012
16	8.0	45.7	23.0	40.0	3402	1843	3415	4012
17	8.0	26.0	21.2	50.0	4158	2784	4773	5563
18	17.0	25.0	31.2	53.0	6240	3562	5209	6693
19	10.0	28.5	37.3	60.0	6987	4032	6274	7324
20	10.0	67.3	54.7	60.0	6668	4032	6274	7324
21	8.0	25.5	30.0	60.0	5941	3917	6274	7165
22	12.0	24.0	42.4	80.0	8655	7066	9660	10716
23	12.0	24.0	45.5	80.0	9288	7066	9660	10716
24	25.0	25.0	60.5	83.0	12100	8605	10208	12456
25	16.0	35.0	82.2	100.0	13894	11136	13500	14107
26	16.0	37.7	120.9	125.0	19690	16920	18867	17221
27	20.0	28.3	183.2	170.0	34438	31008	29923	31008
28	24.0	42.0	258.8	200.0	39934	43008	38184	43008
29	24.0	33.2	235.3	220.0	40837	51533	44052	51533
30	Embedment Depth * 2 <= Edge Distance							
31	6.0	22.0	5.9	17.6	1258	399	997	1040
32	6.0	49.0	8.2	17.6	1171	399	997	1040
33	8.0	25.0	6.6	25.0	1320	792	1688	1932
34	8.0	25.0	8.2	25.0	1640	792	1688	1932
35	8.0	25.0	7.5	25.0	1500	792	1688	1932
36	6.0	22.0	10.0	25.0	2132	744	1688	1854
37	6.0	49.0	15.4	25.0	2200	744	1688	1854
38	8.0	11.2	3.3	25.0	986	792	1688	1932
39	8.0	22.3	5.6	25.0	1186	792	1688	1932
40	8.0	12.5	6.1	25.0	1725	792	1688	1932
41	8.0	24.6	7.2	25.0	1452	792	1688	1932

	A	B	C	D	E	F	G	H
42	8.0	29.6	10.2	25.0	1875	792	1688	1932
43	8.0	42.7	10.6	25.0	1622	792	1688	1932
44	8.0	50.7	13.5	25.0	1896	792	1688	1932
45	9.0	22.0	10.0	26.6	2132	909	1852	2174
46	9.0	49.0	13.8	26.6	1971	909	1852	2174
47	6.0	16.4	6.3	28.0	1556	914	2000	2226
48	6.0	20.5	8.1	28.0	1789	914	2000	2226
49	10.0	45.8	11.3	28.0	1670	1021	2000	2400
50	6.0	59.5	12.4	28.0	1608	914	2000	2226
51	10.0	56.8	15.3	28.0	2030	1021	2000	2400
52	10.0	25.0	9.6	30.0	1920	1152	2218	2671
53	10.0	25.0	11.3	30.0	2260	1152	2218	2671
54	10.0	25.0	11.4	30.0	2280	1152	2218	2671
55	10.0	13.1	5.6	30.0	1547	1152	2218	2671
56	10.0	22.4	9.8	30.0	2071	1152	2218	2671
57	10.0	20.6	10.0	30.0	2203	1152	2218	2671
58	10.0	18.5	11.8	30.0	2743	1152	2218	2671
59	10.0	26.4	13.7	30.0	2666	1152	2218	2671
60	10.0	39.1	17.9	30.0	2863	1152	2218	2671
61	12.0	10.9	6.7	31.0	2029	1280	2330	2904
62	12.0	14.3	8.8	31.0	2327	1280	2330	2904
63	12.0	18.9	9.0	31.0	2070	1280	2330	2904
64	12.0	26.7	10.8	31.0	2090	1280	2330	2904
65	12.0	29.9	15.1	31.0	2761	1280	2330	2904
66	12.0	43.1	19.5	31.0	2970	1280	2330	2904
67	12.0	63.8	20.3	31.0	2541	1280	2330	2904
68	6.0	23.2	9.1	32.0	1889	1167	2444	2755
69	10.0	24.0	12.9	36.0	2633	1590	2916	3527
70	10.0	59.0	21.9	36.0	2951	1590	2916	3527
71	10.0	24.0	16.1	36.0	3286	1590	2916	3527
72	10.0	59.0	22.2	36.0	2890	1590	2916	3527
73	10.0	24.0	16.9	36.0	3450	1590	2916	3527
74	10.0	59.0	24.6	36.0	3203	1590	2916	3527
75	10.0	24.0	18.4	36.0	3756	1590	2916	3527
76	10.0	59.0	22.0	36.0	2864	1590	2916	3527
77	10.0	13.0	10.4	36.0	2884	1590	2916	3527
78	10.0	38.0	16.9	36.0	2742	1590	2916	3527
79	10.0	28.0	17.0	36.0	3213	1590	2916	3527
80	10.0	55.0	23.5	36.0	3169	1590	2916	3527
81	8.0	25.0	15.0	36.5	3000	1559	2977	3493
82	8.0	56.0	22.3	36.5	2980	1559	2977	3493

	A	B	C	D	E	F	G	H
83	8.0	25.0	15.5	36.5	3100	1559	2977	3493
84	8.0	56.0	24.2	36.5	3234	1559	2977	3493
85	8.0	25.0	17.6	36.5	3520	1559	2977	3493
86	8.0	56.0	25.5	36.5	3408	1559	2977	3493
87	8.0	25.0	17.5	36.5	3500	1559	2977	3493
88	6.0	56.0	25.7	36.5	3434	1559	2977	3493
89	8.0	13.0	13.1	36.5	3633	1559	2977	3493
90	8.0	28.0	16.2	36.5	3062	1559	2977	3493
91	8.0	38.0	16.3	36.5	2644	1559	2977	3493
92	8.0	56.0	20.3	36.5	2713	1559	2977	3493
93	6.0	24.0	13.9	37.0	2837	1527	3038	3456
94	6.0	24.0	16.1	37.0	3286	1527	3038	3456
95	6.0	24.0	15.1	37.0	3082	1527	3038	3456
96	12.0	25.0	15.8	37.0	3160	1740	3038	3785
97	6.0	24.0	16.9	37.0	3450	1527	3038	3456
98	12.0	25.0	16.2	37.0	3240	1740	3038	3785
99	6.0	24.0	17.1	37.0	3491	1527	3038	3456
100	12.0	25.0	16.8	37.0	3360	1740	3038	3785
101	6.0	13.0	9.5	37.0	2635	1527	3038	3456
102	12.0	12.5	11.7	37.0	3309	1740	3038	3785
103	14.0	22.9	14.5	37.0	3030	1812	3038	3895
104	6.0	28.0	15.7	37.0	2967	1527	3038	3456
105	6.0	38.0	16.1	37.0	2612	1527	3038	3456
106	12.0	26.4	19.6	37.0	3815	1740	3038	3785
107	14.0	63.5	23.0	37.0	2886	1812	3038	3895
108	10.0	25.0	13.6	38.0	2720	1751	3162	3825
109	10.0	25.6	11.4	38.0	2253	1751	3162	3825
110	14.0	13.8	11.9	38.0	3203	1897	3162	4049
111	14.0	20.5	13.0	38.0	2871	1897	3162	4049
112	14.0	25.0	15.7	38.0	3140	1897	3162	4049
113	14.0	27.2	19.2	38.0	3681	1897	3162	4049
114	14.0	36.0	19.9	38.0	3317	1897	3162	4049
115	14.0	44.1	23.1	38.0	3479	1897	3162	4049
116	14.0	63.1	25.9	38.0	3261	1897	3162	4049
117	13.0	20.7	14.7	39.0	3231	1947	3288	4148
118	13.0	24.0	17.5	39.0	3572	1947	3288	4148
119	13.0	49.0	27.9	39.0	3986	1947	3288	4148
120	9.0	22.0	19.8	40.0	4221	1882	3415	4070
121	9.0	49.0	34.1	40.0	4871	1882	3415	4070
122	10.0	15.7	10.6	40.0	2675	1920	3415	4128
123	10.0	21.9	10.8	40.0	2308	1920	3415	4128

	A	B	C	D	E	F	G	H
124	12.0	13.1	11.5	40.0	3177	1997	3415	4245
125	12.0	20.1	13.2	40.0	2944	1997	3415	4245
126	12.0	20.6	14.9	40.0	3283	1997	3415	4245
127	10.0	39.9	15.5	40.0	2454	1920	3415	4128
128	12.0	20.6	16.3	40.0	3591	1997	3415	4245
129	12.0	30.5	16.8	40.0	3042	1997	3415	4245
130	12.0	26.2	18.4	40.0	3595	1997	3415	4245
131	12.0	21.8	18.6	40.0	3984	1997	3415	4245
132	12.0	26.2	19.8	40.0	3868	1997	3415	4245
133	12.0	47.1	21.4	40.0	3118	1997	3415	4245
134	12.0	72.0	24.6	40.0	2899	1997	3415	4245
135	8.0	30.1	22.9	40.0	4174	1843	3415	4012
136	8.0	30.1	23.6	40.0	4302	1843	3415	4012
137	8.0	64.1	24.0	40.0	2998	1843	3415	4012
138	8.0	64.1	27.4	40.0	3422	1843	3415	4012
139	6.0	31.0	14.5	40.0	2604	1766	3415	3895
140	7.0	35.0	25.3	40.0	4276	1805	3415	3953
141	7.0	35.0	25.3	40.0	4276	1805	3415	3953
142	8.0	26.5	16.9	40.0	3283	1843	3415	4012
143	8.0	26.5	16.9	40.0	3283	1843	3415	4012
144	8.0	29.4	18.2	40.0	3357	1843	3415	4012
145	8.0	29.4	18.2	40.0	3357	1843	3415	4012
146	8.0	23.2	12.6	42.0	2616	2016	3675	4315
147	10.0	21.9	12.9	44.0	2757	2281	3940	4748
148	10.0	27.5	14.8	44.0	2822	2281	3940	4748
149	16.0	21.5	18.2	45.0	3925	2635	4075	5289
150	17.0	20.7	21.2	45.0	4660	2678	4075	5353
151	17.0	24.0	27.6	45.0	5634	2678	4075	5353
152	17.0	49.0	40.3	45.0	5757	2678	4075	5353
153	15.0	25.0	22.3	46.0	4460	2694	4212	5389
154	15.0	25.0	27.7	46.0	5540	2694	4212	5389
155	15.0	25.0	26.7	46.0	5340	2694	4212	5389
156	15.0	25.0	22.0	46.0	4400	2694	4212	5389
157	15.0	25.0	27.8	46.0	5560	2694	4212	5389
158	15.0	12.4	17.2	46.0	4884	2694	4212	5389
159	15.0	28.8	30.6	46.0	5702	2694	4212	5389
160	15.0	36.3	36.0	46.0	5975	2694	4212	5389
161	15.0	49.5	43.3	46.0	6154	2694	4212	5389
162	6.0	14.7	9.8	47.0	2556	2391	4350	4957
163	12.0	8.8	11.0	47.0	3708	2662	4350	5354
164	12.0	22.7	13.1	47.0	2750	2662	4350	5354

	A	B	C	D	E	F	G	H
165	12.0	18.7	13.8	47.0	3191	2662	4350	5354
166	12.0	22.7	15.1	47.0	3169	2662	4350	5354
167	12.0	22.6	15.8	47.0	3324	2662	4350	5354
168	12.0	25.6	19.9	47.0	3933	2662	4350	5354
169	12.0	37.6	20.8	47.0	3392	2662	4350	5354
170	12.0	39.9	26.6	47.0	4211	2662	4350	5354
171	15.0	13.1	16.4	50.0	4531	3120	4773	6048
172	15.0	22.4	19.1	50.0	4036	3120	4773	6048
173	15.0	24.1	19.2	50.0	3911	3120	4773	6048
174	13.0	24.0	24.9	50.0	5083	3024	4773	5910
175	15.0	24.1	25.8	50.0	5255	3120	4773	6048
176	8.0	28.2	24.6	50.0	4632	2784	4773	5563
177	8.0	28.2	27.2	50.0	5122	2784	4773	5563
178	8.0	60.7	30.5	50.0	3915	2784	4773	5563
179	8.0	60.7	30.8	50.0	3953	2784	4773	5563
180	8.0	31.0	20.7	50.0	3718	2784	4773	5563
181	8.0	35.0	26.6	50.0	4496	2784	4773	5563
182	10.0	23.2	22.0	51.0	4568	2987	4917	5863
183	12.0	25.0	19.1	53.0	3820	3307	5209	6331
184	12.0	17.3	14.8	53.0	3558	3307	5209	6331
185	17.0	11.3	19.1	53.0	5682	3562	5209	6693
186	17.0	14.4	22.7	53.0	5982	3562	5209	6693
187	17.0	22.0	26.6	53.0	5671	3562	5209	6693
188	17.0	29.1	30.8	53.0	5710	3562	5209	6693
189	17.0	45.5	44.6	53.0	6612	3562	5209	6693
190	20.0	25.0	26.5	55.0	5300	3960	5507	7254
191	20.0	25.0	30.5	55.0	6100	3960	5507	7254
192	20.0	25.0	29.4	55.0	5880	3960	5507	7254
193	10.0	20.5	12.6	55.0	2783	3432	5507	6510
194	10.0	20.9	18.0	55.0	3937	3432	5507	6510
195	12.0	26.1	20.5	55.0	4013	3538	5507	6659
196	14.0	22.5	22.2	55.0	4680	3643	5507	6808
197	20.0	24.4	29.5	55.0	5972	3960	5507	7254
198	14.0	26.2	29.8	55.0	5822	3643	5507	6808
199	20.0	26.4	42.2	55.0	8213	3960	5507	7254
200	14.0	55.4	42.4	55.0	5697	3643	5507	6808
201	14.0	70.3	48.5	55.0	5784	3643	5507	6808
202	20.0	50.7	57.3	55.0	8047	3960	5507	7254
203	8.0	23.2	19.2	55.0	3986	3326	5507	6361
204	14.0	22.7	17.4	57.0	3652	3885	5810	7141
205	20.0	19.2	27.7	57.0	6322	4213	5810	7599

	A	B	C	D	E	F	G	H
206	20.0	48.8	40.4	57.0	5783	4213	5310	7599
207	20.0	63.5	48.9	57.0	6137	4213	5810	7599
208	16.0	15.7	24.6	59.0	6208	4248	6118	7630
209	16.0	63.0	37.8	59.0	4762	4248	6118	7630
210	14.0	25.0	27.7	60.0	5540	4262	6274	7641
211	19.0	68.5	50.5	60.0	6102	4550	6274	8037
212	10.0	28.0	35.6	60.0	6728	4032	6274	7324
213	10.0	28.0	35.6	60.0	6728	4032	6274	7324
214	10.0	51.0	43.8	60.0	6133	4032	6274	7324
215	10.0	51.0	43.8	60.0	6133	4032	6274	7324
216	10.0	56.0	48.3	60.0	6454	4032	6274	7324
217	10.0	31.0	33.7	60.0	6053	4032	6274	7324
218	8.0	23.0	27.7	60.0	5776	3917	6274	7165
219	8.0	25.5	29.6	60.0	5862	3917	6274	7165
220	8.0	25.5	30.5	60.0	6040	3917	6274	7165
221	10.0	55.2	50.7	60.0	6824	4032	6274	7324
222	10.0	19.3	22.5	60.0	5122	4032	6274	7324
223	10.0	19.3	22.5	60.0	5122	4032	6274	7324
224	10.0	34.0	29.7	60.0	5094	4032	6274	7324
225	10.0	28.5	37.1	60.0	6949	4032	6274	7324
226	10.0	28.5	37.4	60.0	7006	4032	6274	7324
227	10.0	28.5	38.1	60.0	7137	4032	6274	7324
228	10.0	29.6	49.1	60.0	9025	4032	6274	7324
229	15.0	20.3	29.4	61.0	6525	4451	6432	7887
230	15.0	63.0	45.8	61.0	5770	4451	6432	7887
231	15.0	22.0	30.2	62.0	6439	4583	6591	8055
232	15.0	49.0	61.0	62.0	8714	4583	6591	8055
233	21.0	25.0	38.2	63.0	7640	5080	6751	8713
234	21.0	10.8	19.4	63.0	5903	5080	6751	8713
235	21.0	13.8	23.8	63.0	6407	5080	6751	8713
236	21.0	17.8	33.5	63.0	7940	5080	6751	8713
237	21.0	21.9	37.8	63.0	8077	5080	6751	8713
238	21.0	30.8	44.0	63.0	7928	5080	6751	8713
239	21.0	42.8	61.0	63.0	9324	5080	6751	8713
240	10.0	14.7	14.8	65.0	3860	4680	7075	8137
241	10.0	30.5	23.3	65.0	4219	4680	7075	8137
242	20.0	19.5	29.6	65.0	6703	5304	7075	8974
243	20.0	20.5	29.7	65.0	6560	5304	7075	8974
244	20.0	19.5	36.5	65.0	8266	5304	7075	8974
245	20.0	27.1	40.0	65.0	7684	5304	7075	8974
246	20.0	50.0	52.0	65.0	7354	5304	7075	8974

	A	B	C	D	E	F	G	H
247	10.0	28.8	36.4	65.0	6783	4680	7075	8137
248	10.0	55.7	37.1	65.0	4971	4680	7075	8137
249	10.0	28.8	38.1	65.0	7100	4680	7075	8137
250	10.0	28.8	38.1	65.0	7100	4680	7075	8137
251	10.0	55.7	39.7	65.0	5319	4680	7075	8137
252	15.0	23.2	30.5	66.0	6332	5132	7239	8722
253	16.0	25.0	29.0	67.0	5800	5339	7404	8974
254	16.0	8.8	19.2	67.0	6472	5339	7404	8974
255	16.0	22.2	33.1	67.0	7025	5339	7404	8974
256	16.0	32.1	45.5	67.0	8031	5339	7404	8974
257	16.0	72.5	59.9	67.0	7035	5339	7404	8974
258	12.0	22.0	32.2	67.0	6865	5081	7404	8632
259	18.0	22.6	29.9	70.0	6290	5914	7906	9648
260	18.0	25.6	31.6	70.0	6245	5914	7906	9648
261	18.0	37.6	40.7	70.0	6637	5914	7906	9648
262	10.0	48.0	35.3	70.0	5095	5376	7906	8944
263	10.0	36.0	34.8	70.0	5800	5376	7906	8944
264	10.0	48.0	35.6	70.0	5138	5376	7906	8944
265	19.0	25.0	38.4	71.0	7680	6134	8076	9904
266	19.0	17.6	34.3	71.0	8176	6134	8076	9904
267	19.0	24.0	40.5	71.0	8267	6134	8076	9904
268	19.0	24.0	43.8	71.0	8941	6134	8076	9904
269	14.0	20.5	25.5	73.0	5632	6097	8420	9786
270	14.0	23.0	32.8	73.0	6839	6097	8420	9786
271	25.0	25.0	41.9	74.0	8380	7033	8594	10952
272	25.0	25.0	49.9	74.0	9980	7033	8594	10952
273	25.0	25.0	65.0	74.0	13000	7033	8594	10952
274	25.0	12.2	31.3	74.0	8961	7033	8594	10952
275	25.0	18.5	39.3	74.0	9137	7033	8594	10952
276	25.0	29.4	60.5	74.0	11158	7033	8594	10952
277	25.0	56.6	105.0	74.0	13957	7033	8594	10952
278	16.0	25.0	43.7	75.0	8740	6552	8769	10294
279	16.0	25.0	46.4	75.0	9280	6552	8769	10294
280	16.0	25.0	48.3	75.0	9660	6552	8769	10294
281	18.0	20.7	40.1	75.0	8814	6696	8769	10478
282	18.0	29.3	54.2	75.0	10013	6696	8769	10478
283	18.0	40.3	62.6	75.0	9861	6696	8769	10478
284	18.0	39.1	67.6	75.0	10811	6696	8769	10478
285	18.0	57.0	79.3	75.0	10504	6696	8769	10478
286	20.0	23.3	39.1	78.0	8100	7338	9300	11157
287	20.0	25.9	46.6	78.0	9157	7338	9300	11157

	A	B	C	D	E	F	G	H
288	20.0	69.8	68.9	78.0	8247	7338	9300	11157
289	12.0	24.0	45.1	80.0	9206	7066	9660	10716
290	25.0	19.5	48.8	80.0	11051	8064	9660	11962
291	12.0	49.0	75.8	80.0	10829	7066	9660	10716
292	12.0	49.0	76.0	80.0	10857	7066	9660	10716
293	10.0	30.5	46.4	80.0	8402	6912	9660	10525
294	10.0	30.5	46.4	80.0	8402	6912	9660	10525
295	12.0	40.8	52.8	80.0	8266	7066	9660	10716
296	12.0	26.9	45.6	80.0	8792	7066	9660	10716
297	12.0	29.3	53.2	80.0	9828	7066	9660	10716
298	12.0	30.0	38.4	80.0	7011	7066	9660	10716
299	12.0	30.0	40.8	80.0	7449	7066	9660	10716
300	12.0	40.8	49.3	80.0	7718	7066	9660	10716
301	12.0	40.8	51.6	80.0	8078	7066	9660	10716
302	12.0	55.2	75.8	80.0	10202	7066	9660	10716
303	18.0	20.5	40.7	81.0	8989	7698	9842	11452
304	20.0	22.7	32.8	82.0	6884	8029	10024	11806
305	16.0	23.2	49.0	82.0	10173	7715	10024	11417
306	25.0	17.9	41.1	83.0	9714	8605	10208	12456
307	25.0	14.3	41.4	83.0	10948	8605	10208	12456
308	25.0	19.5	55.3	83.0	12523	8605	10208	12456
309	25.0	28.7	67.7	83.0	12637	8605	10208	12456
310	25.0	50.9	76.0	83.0	10653	8605	10208	12456
311	25.0	68.8	92.4	83.0	11140	8605	10208	12456
312	22.0	25.0	48.2	87.0	9640	9104	10955	12800
313	22.0	22.5	47.4	87.0	9993	9104	10955	12800
314	22.0	27.2	53.3	87.0	10220	9104	10955	12800
315	22.0	55.6	102.2	87.0	13706	9104	10955	12800
316	24.0	25.0	58.7	89.0	11740	9655	11335	13318
317	24.0	22.6	46.5	89.0	9781	9655	11335	13318
318	24.0	22.7	50.5	89.0	10599	9655	11335	13318
319	24.0	37.6	62.9	89.0	10258	9655	11335	13318
320	22.0	20.5	51.0	90.0	11264	9677	11527	13269
321	22.0	68.0	90.9	90.0	11023	9677	11527	13269
322	25.0	25.0	53.5	94.0	10700	10739	12303	14194
323	24.0	25.0	58.5	97.0	11700	11268	12897	14539
324	24.0	19.4	53.9	97.0	12237	11268	12897	14539
325	24.0	20.3	70.6	97.0	15670	11268	12897	14539
326	24.0	45.0	87.6	97.0	13059	11268	12897	14539
327	24.0	25.0	77.7	100.0	15540	11904	13500	14978
328	24.0	25.0	80.3	100.0	16060	11904	13500	14978

	A	B	C	D	E	F	G	H
329	24.0	25.0	79.0	100.0	15800	11904	13500	14978
330	25.0	24.5	60.7	100.0	12263	12000	13500	15087
331	24.0	18.3	62.1	100.0	14517	11904	13500	14978
332	25.0	24.6	65.5	100.0	13206	12000	13500	15087
333	24.0	19.9	72.2	100.0	16185	11904	13500	14978
334	24.0	40.3	96.6	100.0	15217	11904	13500	14978
335	25.0	56.2	102.9	100.0	13726	12000	13500	15087
336	24.0	39.1	117.2	100.0	18743	11904	13500	14978
337	24.0	61.7	147.3	100.0	18753	11904	13500	14978
338	12.0	41.0	83.5	105.0	13041	11794	14525	14345
339	28.0	8.8	45.1	105.0	15203	13406	14525	16133
340	28.0	22.7	54.5	105.0	11439	13406	14525	16133
341	25.0	22.7	57.4	105.0	12048	13104	14525	15798
342	28.0	25.0	86.2	105.0	17240	13406	14525	16133
343	28.0	32.3	95.7	105.0	16839	13406	14525	16133
344	28.0	62.2	133.0	105.0	16864	13406	14525	16133
345	28.0	75.4	157.0	105.0	18081	13406	14525	16133
346	28.0	22.7	68.8	114.0	14440	15540	16432	17345
347	16.0	63.0	128.2	123.9	16152	16640	18618	17101
348	16.0	35.0	101.3	124.0	17123	16666	18641	17112
349	16.0	35.0	101.8	124.5	17207	16793	18754	17167
350	16.0	37.7	120.6	125.0	19642	16920	18867	17221
351	28.0	18.3	34.5	125.0	19753	18360	18867	18674
352	28.0	24.2	114.6	125.0	23296	18360	18867	18674
353	28.0	39.1	171.6	125.0	27443	18360	18867	18674
354	28.0	55.5	201.8	125.0	27088	18360	18867	18674
355	16.0	29.4	81.8	125.0	15086	16920	18867	17221
356	16.0	37.7	121.2	125.0	19739	16920	18867	17221
357	16.0	24.8	72.3	125.0	14518	16920	18867	17221
358	16.0	24.8	76.5	125.0	15362	16920	18867	17221
359	16.0	54.9	93.5	125.0	12619	16920	18867	17221
360	16.0	58.4	100.5	125.0	13151	16920	18867	17221
361	32.0	18.3	90.4	148.0	21132	25574	24307	25574
362	32.0	26.1	142.0	148.0	27795	25574	24307	25574
363	32.0	42.9	204.6	148.0	31238	25574	24307	25574
364	32.0	46.6	222.6	148.0	32609	25574	24307	25574
365	20.0	28.3	190.7	170.0	35847	31008	29923	31008
366	20.0	28.3	199.6	170.0	37520	31008	29923	31008
367	24.0	42.0	257.2	220.0	39687	51533	44052	51533
368	24.0	27.6	240.3	220.0	45740	51533	44052	51533
369	d7gert.pri							

	A	B	C	D	E	F	G	H
1	Bolt	Concrete	Actual	Embedment	Normalized	ACI 349-85	KAPPA	VCA
2	Diameter	Strength	Load	Depth	Actual	Equation	Equation	Equation
3	mm	fcc	Capacity	mm	Load	sqrt(N)*mm	sqrt(N)*mm	sqrt(N)*m
4		N/mm^2	KN		Capacity			
5					sqrt(N)*mm			
6								
7	Embedment Depth * 2 > Edge Distance							
8	12.0	23.0	33.0	75.0	6881	6264	8769	9926
9	8.0	23.0	16.0	50.0	3336	2784	4773	5563
10	10.0	25.5	21.0	43.0	4159	2188	3807	4592
11	Embedment Depth * 2 <= Edge Distance							
12	8.9	34.1	10.0	34.2	1712	1415	2700	3207
13	8.0	23.0	15.0	50.0	3128	2784	4773	5563
14	8.0	23.0	25.0	65.0	5213	4555	7075	7970
15	12.0	23.0	29.0	75.0	6047	6264	8769	9926
16	12.0	23.0	57.0	100.0	11885	10752	13500	13671
17	16.0	34.0	43.0	53.0	7374	3511	5209	6620
18	20.0	34.0	54.0	69.0	9261	5895	7738	9655
19	25.0	34.0	68.0	76.0	11662	7369	8944	11292
20	16.0	28.0	41.0	51.0	7748	3280	4917	6285
21	6.0	26.4	7.0	22.0	1362	591	1393	1504
22	8.0	26.4	11.0	24.0	2141	737	1587	1810
23	10.0	26.4	16.0	30.0	3114	1152	2218	2671
24	12.0	26.4	16.0	40.0	3114	1997	3415	4245
25	16.0	26.4	28.0	53.0	5449	3511	5209	6620
26	20.0	26.4	40.0	68.0	7785	5745	7570	9486
27	6.0	25.5	13.0	37.0	2574	1527	3038	3456
28	6.0	32.3	14.0	43.0	2463	2023	3807	4345
29	8.0	24.7	13.0	43.0	2616	2105	3807	4468
30	8.0	32.3	21.0	47.0	3695	2482	4350	5089
31	10.0	24.7	18.0	50.0	3622	2880	4773	5702
32	10.0	24.7	23.0	50.0	4628	2880	4773	5702
33	10.0	32.3	31.0	57.0	5455	3666	5810	6835
34	12.0	32.3	35.0	58.0	6158	3898	5963	7153
35	8.0	32.3	29.0	62.0	5103	4166	6591	7487
36	12.0	23.0	42.0	65.0	8758	4805	7075	8304
37	10.0	32.3	42.0	72.0	7390	5668	8248	9265
38	12.0	32.3	53.0	78.0	9326	6739	9300	10402
39	16.0	23.0	57.0	90.0	11885	9158	11527	12652
40	16.0	23.0	67.0	90.0	13970	9158	11527	12652
41	20.0	25.5	86.0	110.0	17031	13728	15575	15905

	A	B	C	D	E	F	G	H
42	20.0	25.5	87.0	110.0	17229	13728	15575	15905
43	16.0	32.3	93.0	117.0	16364	14939	17085	16312
44	20.0	32.3	113.0	136.0	19883	20367	21411	20367
45	6.0	26.4	9.0	25.0	1752	744	1688	1854
46	6.0	28.1	10.0	25.0	1886	744	1688	1854
47	8.0	26.4	11.0	30.0	2141	1094	2218	2579
48	8.0	28.1	13.0	30.0	2452	1094	2218	2579
49	8.0	26.4	13.0	30.0	2530	1094	2218	2579
50	10.0	26.4	16.0	40.0	3114	1920	3415	4128
51	10.0	28.1	19.0	40.0	3584	1920	3415	4128
52	10.0	29.8	19.0	40.0	3481	1920	3415	4128
53	10.0	26.4	23.0	43.0	4476	2188	3807	4592
54	12.0	26.4	24.0	50.0	4671	2976	4773	5840
55	12.0	28.1	28.0	50.0	5282	2976	4773	5840
56	12.0	29.8	40.0	50.0	7327	2976	4773	5840
57	12.0	26.4	32.0	53.0	6228	3307	5209	6331
58	16.0	26.4	29.0	60.0	5644	4378	6274	7799
59	16.0	29.8	47.0	60.0	8610	4378	6274	7799
60	16.0	28.1	35.0	64.0	6603	4915	6912	8472
61	16.0	26.4	40.0	65.0	7785	5054	7075	8639
62	20.0	26.4	58.0	80.0	11288	7680	9660	11483
63	19.0	28.1	66.0	81.0	12451	7776	9842	11548
64	20.0	26.4	54.0	83.0	10510	8207	10208	11966
65	15.9	35.7	33.9	63.5	5674	4840	6831	8380
66	15.9	35.7	36.4	63.5	6092	4840	6831	8380
67	15.9	35.7	36.4	63.5	6092	4840	6831	8380
68	15.9	35.7	36.4	63.5	6092	4840	6831	8380
69	15.9	51.9	49.6	63.5	6885	4840	6831	8380
70	15.9	51.9	53.9	63.5	7482	4840	6831	8380
71	15.9	51.9	56.4	63.5	7829	4840	6831	8380
72	12.7	28.4	29.8	63.5	5592	4645	6831	8116
73	12.7	28.4	30.3	63.5	5686	4645	6831	8116
74	12.7	28.4	22.3	63.5	4185	4645	6831	8116
75	12.7	28.4	29.6	63.5	5554	4645	6831	8116
76	12.7	37.0	32.7	63.5	5376	4645	6831	8116
77	12.7	38.6	35.9	63.5	5778	4645	6831	8116
78	12.7	38.6	37.0	63.5	5955	4645	6831	8116
79	12.7	38.6	37.0	63.5	5955	4645	6831	8116
80	15.9	16.0	31.6	71.6	7900	6014	8179	9728
81	12.0	47.1	59.0	80.0	8597	7066	9660	10716
82	12.0	35.3	62.0	80.0	10435	7066	9660	10716

	A	B	C	D	E	F	G	H
83	12.0	47.1	54.7	80.0	7970	7066	9660	10716
84	19.5	28.5	66.3	82.6	12419	8096	10135	11854
85	19.5	30.1	57.4	82.6	10462	8096	10135	11854
86	19.5	30.1	69.4	82.6	12650	8096	10135	11854
87	19.5	45.9	63.0	83.0	9299	8167	10208	11917
88	19.5	16.0	37.3	87.4	9325	8969	11031	12611
89	20.0	27.1	157.3	170.0	30216	31008	29923	31008
90	d7eng.pri							

	A	B	C	D	E	F	G	H
1	Bolt	Concrete	Actual	Embedment	Normalized	ACI 349-85	KAPPA	VCA
2	Diameter	Strength	Load	Depth	Actual	Equation	Equation	Equation
3	mm	fcc	Capacity	mm	Load	sqrt(N)*mm	sqrt(N)*mm	sqrt(N)*mm
4		N/mm^2	kN		Capacity			
5					sqrt(N)*mm			
6								
7	Embedment Depth * 2 > Edge Distance							
8	6.0	28.5	10.7	27.7	2004	896	1968	2188
9	8.9	28.5	15.6	33.3	2922	1349	2594	3079
10	8.9	18.6	8.9	34.2	2064	1415	2700	3207
11	8.9	34.1	10.0	34.2	1712	1415	2700	3207
12	9.5	17.2	13.8	40.0	3327	1901	3415	4099
13	9.5	17.2	15.2	40.0	3665	1901	3415	4099
14	12.7	26.0	23.8	51.6	4668	3185	5004	6151
15	12.7	25.2	28.0	51.6	5578	3185	5004	6151
16	12.7	40.7	32.7	51.6	5126	3185	5004	6151
17	12.7	25.2	32.9	51.6	6554	3185	5004	6151
18	12.7	28.5	34.7	51.6	6500	3185	5004	6151
19	12.7	25.2	36.6	51.6	7291	3185	5004	6151
20	12.7	40.7	40.9	51.6	6411	3185	5004	6151
21	12.7	50.4	44.9	51.6	6325	3185	5004	6151
22	12.7	50.4	46.3	51.6	6522	3185	5004	6151
23	12.7	40.7	48.4	51.6	7587	3185	5004	6151
24	12.7	50.4	49.8	51.6	7015	3185	5004	6151
25	12.7	26.0	30.2	51.6	5923	3185	5004	6151
26	15.9	28.5	46.7	62.7	8748	4731	6702	8245
27	22.2	28.5	79.2	93.7	14836	10425	12245	13854
28	Embedment Depth * 2 <= Edge Distance							
29	6.0	32.3	7.8	25.4	1372	766	1728	1902
30	6.0	32.3	8.7	25.4	1531	766	1728	1902
31	6.0	32.3	9.3	25.4	1636	766	1728	1902
32	9.5	16.5	9.1	26.0	2240	886	1790	2118
33	6.0	45.9	12.7	28.0	1875	914	2000	2226
34	6.0	16.5	6.6	28.9	1625	968	2097	2342
35	6.0	32.3	8.5	31.8	1496	1154	2421	2728
36	6.0	32.3	10.5	31.8	1848	1154	2421	2728
37	9.5	16.1	10.9	35.5	2717	1534	2855	3427
38	9.5	34.5	13.6	37.8	2315	1716	3137	3767
39	10.0	28.5	22.7	38.9	4252	1826	3275	3961
40	9.5	18.6	14.4	38.9	3339	1807	3275	3933
41	10.0	45.9	16.7	39.0	2465	1835	3288	3976

	A	B	C	D	E	F	G	H
42	9.5	34.5	14.1	39.3	2401	1841	3326	3993
43	8.9	31.8	6.9	40.0	1224	1878	3415	4064
44	8.9	31.8	7.9	40.0	1401	1878	3415	4064
45	8.9	31.8	8.0	40.0	1419	1878	3415	4064
46	8.9	31.8	8.5	40.0	1507	1878	3415	4064
47	8.9	31.8	9.8	40.0	1738	1878	3415	4064
48	6.0	30.4	15.0	41.3	2721	1875	3583	4089
49	12.7	18.0	21.0	50.8	4950	3097	4888	6020
50	12.7	18.0	24.0	50.8	5657	3097	4888	6020
51	12.7	26.0	31.7	51.6	6217	3185	5004	6151
52	12.7	26.0	29.1	51.6	5707	3185	5004	6151
53	12.7	26.0	27.9	51.6	5472	3185	5004	6151
54	12.7	26.0	32.5	51.6	6374	3185	5004	6151
55	12.7	45.9	35.0	52.0	5166	3230	5062	6217
56	8.9	34.0	30.7	57.0	5265	3606	5810	6751
57	15.9	31.6	33.4	57.0	5942	3989	5810	7286
58	12.7	32.6	32.6	57.5	5710	3875	5886	7124
59	12.0	17.1	21.3	58.2	5151	3922	5994	7186
60	12.0	29.5	23.2	58.9	4271	4009	6102	7301
61	12.0	15.3	19.0	60.2	4857	4173	6306	7515
62	12.0	32.6	23.4	60.7	4098	4236	6384	7598
63	12.0	16.1	19.0	60.8	4735	4249	6400	7614
64	15.9	35.7	33.9	63.5	5674	4840	6831	8380
65	15.9	35.7	36.4	63.5	6092	4840	6831	8380
66	15.9	35.7	36.4	63.5	6092	4840	6831	8380
67	15.9	35.7	36.4	63.5	6092	4840	6831	8380
68	15.9	51.9	49.6	63.5	6805	4840	6831	8380
69	15.9	51.9	53.9	63.5	7482	4840	6831	8380
70	15.9	51.9	56.4	63.5	7829	4840	6831	8380
71	12.7	28.4	29.8	63.5	5592	4645	6831	8116
72	12.7	28.4	30.3	63.5	5686	4645	6831	8116
73	12.7	28.4	22.3	63.5	4185	4645	6831	8116
74	12.7	28.4	29.6	63.5	5554	4645	6831	8116
75	12.7	37.0	32.7	63.5	5376	4645	6831	8116
76	12.7	38.6	35.9	63.5	5778	4645	6831	8116
77	12.7	38.6	37.0	63.5	5955	4645	6831	8116
78	12.7	38.6	37.0	63.5	5955	4645	6831	8116
79	15.9	16.0	31.6	71.6	7900	6014	8179	9728
80	12.0	47.1	59.0	80.0	8597	7066	9660	10716
81	12.0	35.3	62.0	80.0	10435	7066	9660	10716
82	12.0	47.1	54.7	80.0	7970	7066	9660	10716

	A	B	C	D	E	F	G	H
83	19.5	28.5	66.3	82.6	12419	8096	10135	11854
84	19.5	30.1	57.4	82.6	10462	8096	10135	11854
85	19.5	30.1	69.4	82.6	12650	8096	10135	11854
86	19.5	45.9	63.0	83.0	9299	8167	10208	11917
87	19.5	16.0	37.3	87.4	9325	8969	11031	12611
88	20.0	27.1	157.3	170.0	30216	31008	29923	31008
89	d6usa.pri							

	A	B	C	D	E	F	G	H
1	Bolt	Concrete	Actual	Embedment	Normalized	ACI 349-85	KAPPA	VCA
2	Diameter	Strength	Load	Depth	Actual	Equation	Equation	Eqaution
3	mm	fcc	Capacity	mm	Load	sqrt(N)*mm	sqrt(N)*mm	sqrt(N)*mm
4		N/mm^2	KN		Capacity			
5					sqrt(N)*mm			
6								
7	Embedment Depth * 2 <= Edge Distance							
8	8.0	18.1	13.8	42.0	3244	2016	3675	4315
9	8.0	18.2	20.6	70.0	4829	5242	7906	8769
10	8.0	18.8	18.7	53.0	4313	3104	5209	6041
11	8.0	18.8	19.4	58.0	4474	3675	5963	6843
12	8.0	21.2	19.7	61.0	4279	4041	6432	7326
13	8.0	22.0	21.7	61.0	4626	4041	6432	7326
14	8.0	26.5	24.8	63.5	4818	4359	6831	7729
15	8.0	26.5	24.8	63.5	4818	4359	6831	7729
16	8.0	26.5	26.8	63.5	5206	4359	6831	7729
17	8.0	26.5	26.8	63.5	5206	4359	6831	7729
18	8.0	27.9	25.4	62.5	4809	4230	6670	7568
19	8.0	31.7	33.5	61.0	5950	4041	6432	7326
20	8.0	35.0	23.0	53.0	3888	3104	5209	6041
21	8.0	35.0	26.2	58.0	4429	3675	5963	6843
22	8.0	35.0	28.7	63.0	4851	4294	6751	7648
23	8.0	37.1	20.3	42.0	3333	2016	3675	4315
24	10.0	24.7	30.5	70.0	6137	5376	7906	8944
25	10.0	24.7	32.6	70.0	6559	5376	7906	8944
26	10.0	26.5	33.5	73.0	6508	5817	8420	9424
27	10.0	26.5	38.2	73.0	7421	5817	8420	9424
28	10.0	27.9	38.7	71.0	7327	5521	8076	9105
29	10.0	27.9	38.9	71.0	7365	5521	8076	9105
30	10.0	34.4	47.9	73.0	8167	5817	8420	9424
31	10.0	36.4	49.6	73.0	8221	5817	8420	9424
32	12.0	15.1	28.4	76.5	7309	6499	9033	10165
33	12.0	15.1	26.6	77.0	6845	6579	9122	10244
34	12.0	15.1	28.3	77.0	7283	6579	9122	10244
35	12.0	15.1	32.8	77.0	8441	6579	9122	10244
36	12.0	18.0	24.7	60.0	5822	4147	6274	7482
37	12.0	18.1	44.1	101.0	10366	10956	13703	13808
38	12.0	18.8	35.1	77.0	8095	6579	9122	10244
39	12.0	18.8	38.9	83.0	8972	7570	10208	11182
40	12.0	19.8	40.2	78.0	9034	6739	9300	10402
41	12.0	19.8	41.1	78.0	9237	6739	9300	10402

	A	B	C	D	E	F	G	H
42	12.0	23.7	49.6	76.5	10188	6499	9033	10165
43	12.0	23.7	52.8	78.0	10846	6739	9300	10402
44	12.0	29.0	58.1	78.0	10789	6739	9300	10402
45	12.0	35.0	66.7	77.0	11274	6579	9122	10244
46	12.0	35.0	71.9	82.0	12153	7400	10024	11028
47	12.0	35.0	74.8	87.0	12644	8268	10955	11792
48	12.0	40.0	73.6	100.0	11637	10752	13500	13671
49	12.0	42.0	46.4	60.0	7160	4147	6274	7482
50	12.0	42.0	47.5	60.0	7329	4147	6274	7482
51	12.0	43.7	62.1	77.0	9394	6579	9122	10244
52	12.0	49.7	67.3	76.5	9546	6499	9033	10165
53	16.0	40.8	91.8	101.0	14372	11344	13703	14246
54	16.0	45.8	92.7	101.0	13698	11344	13703	14246
55	16.0	28.4	82.4	101.0	15462	11344	13703	14246
56	16.0	27.4	74.4	101.0	14213	11344	13703	14246
57	16.0	28.4	78.0	101.0	14636	11344	13703	14246
58	16.0	33.0	81.5	101.0	14153	11344	13703	14246
59	16.0	28.7	62.7	101.0	11704	11344	13703	14246
60	20.0	16.7	81.4	106.0	19919	12822	14733	15375
61	20.0	42.0	120.0	106.0	18516	12822	14733	15375
62	20.0	34.7	129.3	120.0	21950	16128	17746	17139
63	20.0	54.7	149.7	120.0	20241	16128	17746	17139
64	20.0	54.9	156.4	120.0	21108	16128	17746	17139
65	20.0	13.6	66.3	125.0	17978	17400	18867	17705
66	20.0	27.4	130.2	125.0	24873	17400	18867	17705
67	20.0	38.4	163.2	125.0	26336	17400	18867	17705
68	20.0	19.0	80.2	126.0	18399	17660	19094	17814
69	20.0	18.5	98.7	126.0	22947	17660	19094	17814
70	20.0	28.2	117.9	126.0	22202	17660	19094	17814
71	20.0	33.3	120.1	126.0	20812	17660	19094	17814
72	20.0	28.2	131.2	126.0	24706	17660	19094	17814
73	20.0	27.9	101.0	126.0	19121	17660	19094	17814
74	20.0	32.5	104.5	126.0	18331	17660	19094	17814
75	20.0	26.3	110.3	126.0	21508	17660	19094	17814
76	20.0	24.9	114.8	136.0	23006	20367	21411	20367
77	20.0	38.4	141.6	137.0	22851	20649	21648	20649
78	20.0	15.8	97.2	146.0	24453	23267	23816	23267
79	20.0	36.1	153.2	147.0	25498	23567	24061	23567
80	20.0	16.7	116.2	156.0	28435	26358	26304	26358
81	20.0	42.0	189.2	156.0	29194	26358	26304	26358
82	8.0	27.9	21.4	60.0	4051	3917	6274	7165

	A	B	C	D	E	F	G	H
83	8.0	31.7	30.3	60.0	5382	3917	6274	7165
84	10.0	24.7	28.9	69.0	5815	5233	7738	8784
85	12.0	19.8	33.5	76.5	7529	6499	9033	10165
86	12.0	49.7	62.8	76.5	8908	6499	9033	10165
87	d7fra.pri							

	A	B	C	D	E	F	G	H
1	Bolt	Concrete	Actual	Embedment	Normalized	ACI 349-85	KAPPA	VCA
2	Diameter	Strength	Load	Depth	Actual	Equation	Equation	Equation
3	mm	fcc	Capacity	mm	Load	sqrt(N)*mm	sqrt(N)*mm	sqrt(N)*mm
4		N/mm^2	KN		Capacity			
5					sqrt(N)*mm			
6								
7	Embedment Depth * 2 <= Edge Distance							
8	10.0	23.2	14.2	32.0	2948	1290	2444	2949
9	10.0	26.7	12.6	32.0	2438	1290	2444	2949
10	8.0	40.5	16.9	35.0	2656	1445	2795	3276
11	8.0	26.0	14.1	36.0	2765	1521	2916	3420
12	10.0	23.2	16.1	36.0	3343	1590	2916	3527
13	8.0	40.6	16.6	37.0	2605	1598	3038	3566
14	8.0	25.7	15.5	38.0	3057	1678	3162	3713
15	10.0	40.6	19.5	39.0	3060	1835	3288	3976
16	10.0	26.0	14.8	41.0	2903	2007	3544	4282
17	10.0	40.5	21.3	41.0	3347	2007	3544	4262
18	6.0	23.1	13.7	43.0	2850	2023	3807	4345
19	6.0	25.8	15.0	44.0	2953	2112	3940	4496
20	16.0	18.2	17.6	46.0	4126	2473	4212	5063
21	16.0	24.9	26.8	48.0	5371	2949	4489	5785
22	16.0	25.7	29.3	51.0	5780	3280	4917	6285
23	10.0	23.4	17.9	55.0	3700	3432	5507	6510
24	12.0	42.6	31.0	57.0	4750	3776	5810	6988
25	16.0	25.7	33.4	58.0	6508	4120	5963	7462
26	8.0	26.9	26.7	60.0	5148	3917	6274	7165
27	10.0	42.6	35.0	63.0	5362	4415	6751	7812
28	10.0	23.1	23.7	64.0	4931	4547	6912	7975
29	16.0	20.7	27.9	65.0	6132	5054	7075	8639
30	16.0	40.5	47.1	67.0	7401	5339	7404	8974
31	12.0	26.9	40.0	72.0	7712	5806	8248	9444
32	12.0	43.7	46.3	72.0	7004	5806	8248	9444
33	16.0	28.9	49.6	103.0	9226	11767	14112	14521
34	16.0	42.8	69.0	104.0	10547	11981	14318	14657
35	16.0	16.3	76.2	125.0	18874	16920	18867	17221
36	d7swe.pri							

	A	B	C	D	E	F	G	H
1	Bolt	Concrete	Actual	Embedment	Normalized	ACI 349-85	KAPPA	VCA
2	Diameter	Strength	Load	Depth	Actual	Equation	Equation	Equation
3	mm	fcc	Capacity	mm	Load	sqrt(N)*mm	sqrt(N)*mm	sqrt(N)*mm
4		N*mm^2	KN		Capacity			
5					sqrt(N)*mm			
6								
7	Embedment Depth * 2 <= Edge Distance							
8	27.0	31	180.0	140.0	32381	22445	25676	22445
9	27.0	31	190.0	142.0	34180	23038	26228	23038
10	30.0	57	206.0	130.0	27357	19968	22975	19968
11	k4swe.pri							

APPENDIX E: SPREADHEET DATA BASE FOR STEEL DATA BASE (U.S. UNITS)

Test Number	d (in.)	do or dh (in.)	hef (in.)	fc (psi)	Actual Capacity (kips)	fy (ksi)	fult (ksi)	Normalized Actual Capacity sq(lb.)*in.	Normalized Predicted Capacity ACI 349-85 sq(lb)*in.
51	1	1.625	12	4300	116	120	150	116000	72689
52	1	1.625	14.1	4245	118	120	150	118000	72689
53	1	1.625	16.2	4200	118	120	150	118000	72689
2a	.625		7	5430	37.5	96	120	37500	19093
2b	.625		7	5430	37.4	96	120	37400	19093
2c	.625		7	5430	37.4	96	120	37400	19093
8a	.625		8	4050	30.9	120	150	30900	23866
8b	.625		8	4050	31.1	120	150	31100	23866
12a	.625		8	4050	31.7	120	150	31700	23866
12b	.625		8	4050	31.2	120	150	31200	23866
15a	.625		8	4050	31.1	120	150	31100	23866
15b	.625		8	4050	31.1	120	150	31100	23866
17a	.625		8	5760	30.9	120	150	30900	23866
17b	.625		8	5760	30.9	120	150	30900	23866
22e	.625		12	4810	31.4	120	150	31400	23866
4a	.625		8	4050	31.1	120	150	31100	23866
5a	.625		8	4050	31.3	120	150	31300	23866
5b	.625		8	4050	31.1	120	150	31100	23866
28a	.625		6	5760	26.3	80	100	26300	15910
28c	.625		6	4520	24.5	80	100	24500	15910
28d	.625		6	4520	23.3	80	100	23300	15910
30a	.625		7	4810	30.6	88	110	30600	17502
30b	.625		7	4810	29.9	88	110	29900	17502
33b	.625		7.5	4520	29.2	120	150	29200	23866
33c	.625		7.5	4520	28.3	120	150	28300	23866
33d	.625		7.5	4520	29.2	120	150	29200	23866
16a	.625		8	5760	30.9	120	150	30900	23866
16b	.625		8	5760	30.8	120	150	30800	23866
21d	.625		7	5130	32.2	120	150	32200	23866
21e	.625		7	5130	32.1	120	150	32100	23866
21f	.625		7	5130	32.1	120	150	32100	23866

APPENDIX G: COMPUTER PROGRAMS FOR CONDUCTING MONTE CARLO ANALYSIS

```
$large
$nofloatcalls
C**   PROGRAM SAFETY3
C*******************************************************
**
C     THIS PROGRAM USES THE MONTE CARLO TECHNIQUE TO
COMPUTE THE
C     PROBABILITY OF FAILURE,UNDER KNOWN LOADS, OF A
SINGLE
TENSILE ANCHOR DESIGNED
C     USING THE APPROACH OF ACI 349 APPENDIX B.  THE
METHOD CAN
BE
C     APPLIED TO ANY DESIGN APPROACH.  THE FOLLOWING
STEPS ARE
C     CARRIED OUT:
C
C     1)    THE LOAD IS ASSUMED TO REPRESENT A
95-PERCENTILE
C           VALUE.  IT IS USED WITH A LOAD FACTOR OF
1.7.
C           BASED ON THIS HISTOGRAM, 1000 LOAD VALUES
ARE
GENERATED.
C           THE NUMBER OF LOAD VALUES CORRESPONDING TO
EACH LOAD
C           DEPENDS ON THE ORDINATES OF THE PROBABILITY
FUNCTION.
C
C     2)    AN ANCHOR IS DESIGNED TO HAVE SUFFICIENT
STEEL AREA
```

```
TO
C             RESIST THE FACTORED LOAD.  THE DESIGN STEEL
STRENGTH
C             IS A FUNCTION OF THE 95-PERCENTILE LOAD,
THE LOAD
C             FACTOR, AND THE PHI FACTOR FOR STEEL.
C
C      3)     THE ANCHOR IS DESIGNED TO HAVE SUFFICIENT
EMBEDMENT
C             TO DEVELOP THE STEEL IN TENSION, USING THE
45 DEGREE
C             CONE FORMULA OF ACI 349 APPENDIX B.
C
C      4)     BASED ON THE RESULTS OF PREVIOUS U.S.
RESEARCH (SEE
C             KLINGNER AND MENDONCA), THE STATISTICAL
DISTRIBUTION
C             OF ACTUAL TO PREDICTED CAPACITY IS KNOWN
FOR STEEL
C             AND ALSO FOR CONCRETE FAILURE.  EACH
DISTRIBUTION IS
C             REPRESENTED BY A SMOOTHED NORMAL
DISTRIBUTION.  AS
C             WITH THE LOADS, 10000 STEEL AND CONCRETE
RESISTANCE
C             VALUES ARE GENERATED.  THE NUMBER OF
RESISTANCE
VALUES
C             CORRESPONDING TO EACH VALUE DEPENDS ON THE
C             STATISTICAL DISTRIBUTION FOR STEEL AND FOR
CONCRETE.
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C
C     5)     THREE GROUPS OF 10000 RANDOM NUMBERS ARE
GENERATED
C            (ZERO TO 999), ONE FOR THE LOAD, ONE FOR
STEEL,
C            AND ONE FOR CONCRETE CAPACITY.  EACH SET OF
THREE
C            RANDOM NUMBERS REPRESENTS SOME COMBINATION
OF LOAD,
C            STEEL STRENGTH, AND CONCRETE STRENGTH.
EACH NUMBER
C            CORRESPONDS RESPECTIVELY TO A PARTICULAR
VALUE
C            ON THE STATISTICAL DISTRIBUTION FOR STEEL
FAILURE
C            AND FOR CONCRETE FAILURE.  BECAUSE THE
NUMBERS ARE
C            RANDOM, THE DISTRIBUTION OF VALUES FOR
STEEL AND
C            CONCRETE FAILURE WILL FOLLOW THE
EXPERIMENTALLY
C            DETERMINED DISTRIBUTIONS FOR EACH TYPE OF
FAILURE.
C
C     6)     BASED ON THOSE VALUES, THE MINIMUM
RESISTANCE (STEEL
C            OR CONCRETE) IS COMPUTED, AND THE QUANTITY
C            (RESISTANCE - LOAD) IS ALSO COMPUTED.
C
C     7)     THE STATISTICAL DISTRIBUTION OF (RESISTANCE
- LOAD),
```

```
C            OBTAINED BY THIS MONTE CARLO TECHNIQUE, IS
ALSO
C            TREATED AS NORMAL.
C            THE NUMBER OF CASES FOR WHICH RESISTANCE IS
LESS THAN
C            LOAD, DIVIDED BY THE TOTAL NUMBER OF CASES,
REPRESENTS
C            THE PROBABILITY OF FAILURE.
C
C*******************************************************
**********
*
      IMPLICIT REAL*8 (A-H,O-Z)
      PARAMETER (NMAX = 10000)
      CHARACTER*15 FILOUT
      COMMON / STAT / AVGLOD, COVLOD, AVGSTL, COVSTL,
STRSTL,
     1               AVGCONC, COVCONC, STRCONC
      COMMON / RAN / NRAN(NMAX,3),DISTLOD(NMAX),
     1              DISTSTL(NMAX), DISTCON(NMAX),
DIFF(NMAX)
      DATA AVGLOD / 1.D0 /
      DATA COVLOD / 0.2D0 /
      DATA AVGSTL / 1.444 /
      DATA COVSTL / 0.156 /
C
C     SET UP OUTPUT FILES
C
      WRITE (*,2001)
 2001 FORMAT (/,5X,'Enter name of output file'/)
      READ (*,1000) FILOUT
```

```
 1000 FORMAT (A15)
      OPEN (6,FILE=FILOUT,STATUS='UNKNOWN')
      write (*,5000)
C
C     FROM CHAPTER 6 TABLE 3 PUT THIS WAY FOR ROW OF
INPUT
C
 5000 format(/,5x,'ENTER MEAN AND COV OF CONCRETE
DATA'/)
      read (*,*) AVGCONC, COVCONC
C
C     GENERATE THREE SETS OF RANDOM NUMBERS
C
      CALL RANDM
C
C     SET UP STATISTICAL DISTRIBUTION OF LOAD.  DESIGN
LOAD
C     IS AT 95% VALUE (1.924 COEFFICIENTS OF VARIATION
AWAY
C     FROM AVERAGE).  THIS SUBROUTINE RETURNS NMAX
VALUES
C     OF LOAD, DISTRIBUTED ACCORDING TO THE ABOVE.
C
      CALL LOAD
C
C     CHECK DISTRIBUTION OF LOADS
C
      KOUNT = 0
      SUM = 0.D0
      SUM2 = 0.D0
      DO 5 I=1,NMAX
```

```
      KOUNT = KOUNT + 1
      SUM = SUM + DISTLOD(I)
      SUM2 = SUM2 + DISTLOD(I)**2
    5 CONTINUE
C
      AVLOD = SUM/DFLOAT(KOUNT)
      STLOD = DSQRT ((SUM2 - SUM**2/DFLOAT(KOUNT))/
     1              DFLOAT(KOUNT - 1))
      COLOD = STLOD/AVLOD
C
C     PRINT CHECK ON DISTRIBUTION OF LOADS
C
      WRITE (6,2002) AVGLOD, COVLOD, AVLOD, COLOD
 2002 FORMAT (1X,// 'TARGET LOADS:',//,
     1        5X, 'AVERAGE LOAD =', F8.3/,
     2        5X, 'COV OF LOADS =',F8.3///,
     3        1X, 'ACTUAL LOADS:',//,
     4        5X, 'AVERAGE LOAD =',F8.3/,
     5        5X, 'COV OF LOADS =', F8.3///)
C
C     SET UP STATISTICAL DISTRIBUTION OF STEEL
RESISTANCES.
C     THIS SUBROUTINE RETURNS NMAX VALUES OF STEEL
C     RESISTANCE, DISTRIBUTED ACCORDING TO THE ABOVE.
C
      CALL RESSTL
C
C     CHECK DISTRIBUTION OF STEEL RESISTANCES
C
      KOUNT = 0
      SUM = 0.D0
```

```
      SUM2 = 0.D0
      DO 10 I=1,NMAX
      KOUNT = KOUNT + 1
      SUM = SUM + DISTSTL(I)
      SUM2 = SUM2 + DISTSTL(I)**2
   10 CONTINUE
C
      AVSTL = SUM/DFLOAT(KOUNT)
      STSTL = DSQRT ((SUM2 - SUM**2/DFLOAT(KOUNT))/
     1            DFLOAT(KOUNT - 1))
      COSTL = STSTL/AVSTL
C
C     PRINT CHECK ON DISTRIBUTION OF STEEL RESISTANCES
C
      WRITE (6,2003) STRSTL, COVSTL, AVSTL, COSTL
 2003 FORMAT (1X,// 'TARGET STEEL RESISTANCE:',//,
     1        5X, 'AVERAGE STEEL RESISTANCE =',F8.3,/,
     2        5X, 'COV OF STEEL RESISTANCES =', F8.3///
     3        1X, 'ACTUAL STEEL RESISTANCES:',//,
     4        5X, 'AVERAGE STEEL RESISTANCE =', F8.3,/,
     5        5X, 'COV OF STEEL RESISTANCES
=',F8.3,///)
C
C     SET UP STATISTICAL DISTRIBUTION OF CONCRETE
RESISTANCES.
C     THIS SUBROUTINE RETURNS NMAX VALUES OF CONCRETE
C     RESISTANCE, DISTRIBUTED ACCORDING TO THE ABOVE.
C
      CALL RESCONC
C
C     CHECK DISTRIBUTION OF CONCRETE RESISTANCES
```

```
C
      KOUNT = 0
      SUM = 0.D0
      SUM2 = 0.D0
      DO 15 I=1,NMAX
      KOUNT = KOUNT + 1
      SUM = SUM + DISTCON(I)
      SUM2 = SUM2 + DISTCON(I)**2
   15 CONTINUE
C
      AVCONC = SUM/DFLOAT(KOUNT)
      STCONC = DSQRT ((SUM2 - SUM**2/DFLOAT(KOUNT))/
     1            DFLOAT(KOUNT - 1))
      COCONC = STCONC/AVCONC
C
C     PRINT CHECK ON DISTRIBUTION OF CONCRETE
RESISTANCES
C
      WRITE (6,2004) STRCONC, COVCONC, AVCONC, COCONC
 2004 FORMAT (1X,// 'TARGET CONCRETE RESISTANCE:',//,
     1        5X, 'AVERAGE CONCRETE RESISTANCE =',
F8.3,/,
     2        5X, 'COV OF CONCRETE RESISTANCES
=',F8.3,///,
     3        1X, 'ACTUAL CONCRETE RESISTANCES:',//,
     4        5X, 'AVERAGE CONCRETE RESISTANCE =',
F8.3/,
     5        5X, 'COV OF CONCRETE RESISTANCES =',
F8.3,///)
C
```

```
C      APPLY MONTE CARLO TECHNIQUE NMAX TIMES.  EACH
TIME,
C      DETERMINE THE LESSER OF STEEL OR CONCRETE
RESISTANCE.
C      CALCULATE (RESISTANCE - LOAD), AND ACCUMULATE THE
C      STATISTICS NECESSARY TO DETERMINE THE MEAN AND
C      STANDARD DEVIATION OF (RESISTANCE - LOAD).
C
      KOUNT = 0
      SUM = 0.D0
      SUM2 = 0.D0
C
      DO 20 I=1,NMAX
      DIFF(I) =
DMIN1(DISTSTL(NRAN(I,2)),DISTCON(NRAN(I,3)))
     1          - DISTLOD(NRAN(I,1))
C
      KOUNT = KOUNT + 1
      SUM = SUM + DIFF(I)
      SUM2 = SUM2 + DIFF(I)**2
   20 CONTINUE
C
      AVGDIFF = SUM/DFLOAT(KOUNT)
      STDDIFF = DSQRT ((SUM2 - SUM**2/DFLOAT(KOUNT))/
     1            DFLOAT(KOUNT - 1))
C
C      GIVEN THE STATISTICAL CHARACTERISTICS OF THE
C      (RESISTANCE - LOAD) CURVE, COMPUTE THE SAFETY
C      INDEX BETA, AND THE PROBABILITY OF FAILURE.
C
      BETA = AVGDIFF/STDDIFF
```

```
      PROB =
AREA(0.D0,AVGDIFF,STDDIFF)/AREA(10.D0,AVGDIFF,STDDIFF)
C
C     WRITE OUTPUT
C
      WRITE (6,2005)
 2005 FORMAT (/,8X,'BETA',10X,'PROBABILITY OF
FAILURE'//)
      WRITE (6,2006) BETA, PROB
 2006 FORMAT (1X,F10.2,15X,E10.3)
      STOP
      END
C*******************************************************
**
      SUBROUTINE RANDM
      IMPLICIT REAL*8 (A-H,O-Z)
      INTEGER*2 IHR, IMIN, ISEC, I100TH
      PARAMETER (NMAX = 10000)
      REAL*4 RANVAL
      COMMON / RAN / NRAN(NMAX,3), DISTLOD(NMAX),
     1              DISTSTL(NMAX), DISTCON(NMAX),
DIFF(NMAX)
C
C     GENERATE 3 DIFFERENT SETS OF
C     PSEUDO-RANDOM NUMBERS, USING A DIFFERENT SEED
EACH TIME.
C
      DO 20 J=1,3
      CALL GETTIM (IHR,IMIN,ISEC,I100TH)
      CALL SEED(I100TH)
      DO 10 I=1,NMAX
```

```
C
C      RANVAL RETURNS A SINGLE RANDOM NUMBER GREATER
THAN OR EQUAL
C      TO ZERO AND LESS THAN 1.0.
C
       CALL RANDOM (RANVAL)
       NRAN(I,J) = INT4(RANVAL*DFLOAT(NMAX))
    10 CONTINUE
    20 CONTINUE
C
       RETURN
       END
C******************************************************
*******
       SUBROUTINE LOAD
       IMPLICIT REAL*8 (A-H,O-Z)
       PARAMETER (NMAX = 10000)
       COMMON / STAT / AVGLOD, COVLOD, AVGSTL, COVSTL,
STRSTL,
     1                AVGCONC, COVCONC, STRCONC
       COMMON / RAN / NRAN(NMAX,3), DISTLOD(NMAX),
     1               DISTSTL(NMAX), DISTCON(NMAX),
DIFF(NMAX)
C
C      SET UP STATISTICAL DISTRIBUTION OF LOADS.
C      COEFFICIENT OF VARIATION DOES NOT CHANGE.
C
       XSTART = AVGLOD*(1.D0 - 5.D0*COVLOD)
       XEND = AVGLOD*(1.D0 + 5.D0*COVLOD)
       DX = (XEND - XSTART)/DFLOAT(NMAX/50 + 1)
       KOUNT = 0
```

```
      SUM = 0.D0
C
      DO 20 I=1,NMAX/50 + 1
      X = XSTART + DX/2.D0 + DFLOAT(I-1)*DX
      SUM = SUM + GAUSS(X,AVGLOD,AVGLOD*COVLOD)
   20 CONTINUE
C
      FACTOR = DFLOAT(NMAX)/SUM
      KOUNT = 0
C
      DO 30 J=1,NMAX/50 + 1
      X = XSTART + DX/2.D0 + DFLOAT(J-1)*DX
      NHIST = INT4(FACTOR *
GAUSS(X,AVGLOD,AVGLOD*COVLOD))
      DO 29 K=1,NHIST
      KOUNT = KOUNT+1
      DISTLOD(KOUNT) = X
   29 CONTINUE
   30 CONTINUE
C
      IF (KOUNT.LT.NMAX) THEN
            DO 35 I=KOUNT+1,NMAX
            DISTLOD(I) = AVGLOD
   35       CONTINUE
      ENDIF
C
      RETURN
      END
C*******************************************************
*******
      SUBROUTINE RESSTL
```

```
      IMPLICIT REAL*8 (A-H,O-Z)
      PARAMETER (NMAX = 10000)
      COMMON / STAT / AVGLOD, COVLOD, AVGSTL, COVSTL,
STRSTL,
     1                AVGCONC, COVCONC, STRCONC
      COMMON / RAN / NRAN(NMAX,3), DISTLOD(NMAX),
     1               DISTSTL(NMAX), DISTCON(NMAX),
DIFF(NMAX)
C
C     SET UP STATISTICAL DISTRIBUTION OF RESISTANCE.
DESIGN
C     RESISTANCE EQUALS DESIGN LOAD TIMES LOAD FACTOR,
DIVIDED
C     BY PHI FACTOR.
C
      DESLOD = AVGLOD*(1.D0 + COVLOD*1.924D0)
      PHISTL = 0.9D0
C
C     DETERMINE DESIGN STRENGTH OF STEEL
C     COEFFICIENT OF VARIATION WILL BE
C     AS GIVEN IN DATA STATEMENTS ABOVE.
C     THIS ASSUMES THAT STEEL DESIGN BASED ON PHI*FY
WILL GOVERN
C     OVER STEEL DESIGN BASED ON 0.8*FUT
C
      STRSTL = (DESLOD*1.7D0/PHISTL)*AVGSTL
C
C     SET UP STATISTICAL DISTRIBUTION OF STEEL
C     RESISTANCES.  COEFFICIENT OF VARIATION
C     DOES NOT CHANGE.
C
```

```
      XSTART = STRSTL*(1.D0 - 5.D0*COVSTL)
      XEND = STRSTL*(1.D0 + 5.D0*COVSTL)
      DX = (XEND - XSTART)/DFLOAT(NMAX/50 + 1)
      KOUNT = 0
      SUM = 0.D0
C
      DO 20 J=1,NMAX/50 + 1
      X = XSTART + DX/2.D0 + DFLOAT(J-1)*DX
      SUM = SUM + GAUSS(X,STRSTL,STRSTL*COVSTL)
   20 CONTINUE
C
      FACTOR = DFLOAT(NMAX)/SUM
      KOUNT = 0
C
      DO 30 J=1,NMAX/50 + 1
      X = XSTART + DX/2.D0 + DFLOAT(J-1)*DX
      NHIST = INT4(FACTOR *
GAUSS(X,STRSTL,STRSTL*COVSTL))
      DO 29 K=1,NHIST
      KOUNT = KOUNT+1
      DISTSTL(KOUNT) = X
   29 CONTINUE
   30 CONTINUE
C
      IF (KOUNT.LT.NMAX) THEN
            DO 35 I=KOUNT+1,NMAX
            DISTSTL(I) = STRSTL
   35       CONTINUE
      ENDIF
C
      RETURN
```

```
      END
C****************************************************
*********
      SUBROUTINE RESCONC
      IMPLICIT REAL*8 (A-H,O-Z)
      PARAMETER (NMAX = 10000)
      COMMON / STAT / AVGLOD, COVLOD, AVGSTL, COVSTL,
STRSTL,
     1                 AVGCONC, COVCONC, STRCONC
      COMMON / RAN / NRAN(NMAX,3), DISTLOD(NMAX),
     1                DISTSTL(NMAX), DISTCON(NMAX),
DIFF(NMAX)
C
C     SET UP STATISTICAL DISTRIBUTION OF CONCRETE
RESISTANCE.
C     DESIGN RESISTANCE EQUALS THE DESIGN STEEL
STRENGTH
C     (DESLOD/PHISTEEL), MULTIPLIED BY AVERAGE RATIO OF
STEEL
C     ULTIMATE STRENGTH TO STEEL YIELD STRENGTH,
DIVIDED
C     BY THE PHI FACTOR FOR CONCRETE, AND MULTIPLIED BY
THE
C     AVERAGE RATIO OF ACTUAL CONCRETE STRENGTH TO
COMPUTED
C     CONCRETE STRENGTH.  THE AVERAGE RATIO OF STEEL
ULTIMATE
C     STRENGTH TO STEEL YIELD STRENGTH IS
CONSERVATIVELY
C     TAKEN AS 1.20.
C
```

```
      PHISTL = 0.9D0
      PHICONC = 0.65D0
C
C     DETERMINE DESIGN STRENGTH OF CONCRETE
C     COEFFICIENT OF VARIATION WILL BE
C     AS GIVEN IN DATA STATEMENTS ABOVE.
C
      DESLOD = AVGLOD*(1.D0 + COVLOD*1.924D0)
      STRCONC =
((DESLOD*1.7D0)/PHISTL)*1.2D0*AVGCONC/PHICONC
C
C     SET UP STATISTICAL DISTRIBUTION OF CONCRETE
C     RESISTANCES.  COEFFICIENT OF VARIATION
C     DOES NOT CHANGE.
C
      XSTART = STRCONC*(1.D0 - 5.D0*COVCONC)
      XEND = STRCONC*(1.D0 + 5.D0*COVCONC)
      DX = (XEND - XSTART)/DFLOAT(NMAX/50 + 1)
      KOUNT = 0
      SUM = 0.D0
C
      DO 20 J=1,NMAX/50 + 1
      X = XSTART + DX/2.D0 + DFLOAT(J-1)*DX
      SUM = SUM + GAUSS(X,STRCONC,STRCONC*COVCONC)
   20 CONTINUE
C
      FACTOR = DFLOAT(NMAX)/SUM
      KOUNT = 0
      SUM = 0.D0
C
      DO 30 J=1,NMAX/50 + 1
```

```
      X = XSTART + DX/2.D0 + DFLOAT(J-1)*DX
      NHIST = INT4(FACTOR *
GAUSS(X,STRCONC,STRCONC*COVCONC))
      DO 29 K=1,NHIST
      KOUNT = KOUNT+1
      DISTCON(KOUNT) = X
   29 CONTINUE
   30 CONTINUE
C
      IF (KOUNT.LT.NMAX) THEN
            DO 35 I=KOUNT+1,NMAX
            DISTCON(I) = STRCONC
   35       CONTINUE
      ENDIF
C
      RETURN
      END
C*******************************************************
*********
      FUNCTION AREA(XEND,AVG,STD)
C
C     THIS FUNCTION SUBPROGRAM COMPUTES THE AREA UNDER
A GAUSSIAN

C     CURVE BETWEEN -10 AND XEND.
C
      IMPLICIT REAL*8 (A-H,O-Z)
C
C     SET UP LIMITS OF NUMERICAL INTEGRATION
      START = -10.D0
      DX   = (XEND - START)/1000.D0
```

```
      AREA = 0.D0
C
      DO 10 N=1,1000
      X1 = START + DFLOAT(N-1)*DX
      X2 = X1 + DX
      DA = DX*(GAUSS(X1,AVG,STD) +
GAUSS(X2,AVG,STD))/2.D0
      AREA = AREA + DA
   10 CONTINUE
C
      RETURN
      END
C*******************************************************
**********
      FUNCTION GAUSS(X,AVG,STD)
      IMPLICIT REAL*8 (A-H,O-Z)
C
C     THIS FUNCTION SUBPROGRAM RETURNS THE VALUE OF A
GAUSSION
CURVE
C     FOR ANY AVERAGE (AVG) AND STANDARD DEVIATION
(STD).
C
      PI = 2.D0*DASIN(1.D0)
      GAUSS = 1.D0/(STD*DSQRT(2.D0*PI))*
     1        DEXP(-((X - AVG)**2)/(2.D0*STD**2))
C
      RETURN
      END
```

```
$large
$nofloatcalls
C**   PROGRAM SAFETY4
C*******************************************************
**
C      THIS PROGRAM USES THE MONTE CARLO TECHNIQUE TO
COMPUTE THE
C      PROBABILITY OF CONCRETE FAILURE,UNDER UNLIMITED
LOADS, OF A SINGLE TENSILE ANCHOR
C      DESIGNED USING THE APPROACH OF ACI 349 APPENDIX
B.
C      THE METHOD CAN BE APPLIED TO ANY DESIGN APPROACH.
C      THE FOLLOWING STEPS ARE CARRIED OUT:
C
C      1)     AN ANCHOR IS DESIGNED TO HAVE SUFFICIENT
STEEL AREA
TO
C             RESIST SOME LOAD.
C
C      2)     THE ANCHOR IS DESIGNED TO HAVE SUFFICIENT
EMBEDMENT
C             TO DEVELOP THE STEEL IN TENSION, USING THE
45 DEGREE
C             CONE FORMULA OF ACI 349 APPENDIX B.
C
C      3)     BASED ON THE RESULTS OF PREVIOUS U.S.
RESEARCH (SEE
C             KLINGNER AND MENDONCA), THE STATISTICAL
DISTRIBUTION
C             OF ACTUAL TO PREDICTED CAPACITY IS KNOWN
FOR STEEL
```

```
C             AND ALSO FOR CONCRETE FAILURE.  EACH
DISTRIBUTION IS
C             REPRESENTED BY A SMOOTHED NORMAL
DISTRIBUTION.  AS
C             WITH THE LOADS, 10000 STEEL AND CONCRETE
RESISTANCE
C             VALUES ARE GENERATED.  THE NUMBER OF
RESISTANCE
VALUES
C             CORRESPONDING TO EACH VALUE DEPENDS ON THE
C             STATISTICAL DISTRIBUTION FOR STEEL AND FOR
CONCRETE.
C
C      4)     TWO GROUPS OF 10000 RANDOM NUMBERS ARE
GENERATED
C             (ZERO TO 999), ONE FOR STEEL, AND ONE FOR
CONCRETE
C             CAPACITY.  EACH SET OF TWO RANDOM NUMBERS
REPRESENTS
C             SOME COMBINATION OF STEEL STRENGTH, AND
CONCRETE
STRENGTH.
C             EACH NUMBER CORRESPONDS RESPECTIVELY TO A
PARTICULAR
VALUE
C             ON THE STATISTICAL DISTRIBUTION FOR STEEL
FAILURE
C             AND FOR CONCRETE FAILURE.  BECAUSE THE
NUMBERS ARE
C             RANDOM, THE DISTRIBUTION OF VALUES FOR
STEEL AND
```

```
C             CONCRETE FAILURE WILL FOLLOW THE
EXPERIMENTALLY
C             DETERMINED DISTRIBUTIONS FOR EACH TYPE OF
FAILURE.
C
C      5)     BASED ON THOSE VALUES, DISCRETE HISTOGRAMS
ARE
C             GENERATED FOR STEEL AND CONCRETE
RESISTANCE,
C             AND THE QUANTITY (CONCRETE - STEEL) IS
COMPUTED.
C
C      6)     THE STATISTICAL DISTRIBUTION OF (CONCRETE -
STEEL),
C             OBTAINED BY THIS MONTE CARLO TECHNIQUE, IS
ALSO
C             TREATED AS NORMAL.  THE NUMBER OF CASES FOR
WHICH
C             CONCRETE RESISTANCE IS LESS THAN STEEL
RESISTANCE,
C             DIVIDED BY THE TOTAL NUMBER OF CASES,
REPRESENTS
C             THE PROBABILITY OF BRITTLE FAILURE.
C
C*******************************************************
**********
*
      IMPLICIT REAL*8 (A-H,O-Z)
      PARAMETER (NMAX = 10000)
      CHARACTER*15 FILOUT
```

```
      COMMON / STAT / AVGLOD, COVLOD, AVGSTL, COVSTL,
STRSTL,
     1                AVGCONC, COVCONC, STRCONC
      COMMON / RAN / NRAN(NMAX,2),DISTLOD(NMAX),
     1               DISTSTL(NMAX), DISTCON(NMAX),
DIFF(NMAX)
      DATA AVGLOD / 1.D0 /
      DATA COVLOD / 0.2D0 /
      DATA AVGSTL / 1.444 /
      DATA COVSTL / 0.156 /
C
C     SET UP OUTPUT FILES
C
      WRITE (*,2001)
 2001 FORMAT (/,5X,'Enter name of output file'/)
      READ (*,1000) FILOUT
 1000 FORMAT (A15)
      OPEN (6,FILE=FILOUT,STATUS='UNKNOWN')
      write(*,5000)
 5000 format(/,5x,'enter mean and cov of concrete'/)
      read(*,*) avgconc,covconc
C
C     GENERATE TWO SETS OF RANDOM NUMBERS
C
      CALL RANDM
C
C     SET UP STATISTICAL DISTRIBUTION OF STEEL
RESISTANCES.
C     THIS SUBROUTINE RETURNS NMAX VALUES OF STEEL
C     RESISTANCE, DISTRIBUTED ACCORDING TO THE ABOVE.
C
```

```
      CALL RESSTL
C
C     CHECK DISTRIBUTION OF STEEL RESISTANCES
C
      KOUNT = 0
      SUM = 0.D0
      SUM2 = 0.D0
      DO 10 I=1,NMAX
      KOUNT = KOUNT + 1
      SUM = SUM + DISTSTL(I)
      SUM2 = SUM2 + DISTSTL(I)**2
   10 CONTINUE
C
      AVSTL = SUM/DFLOAT(KOUNT)
      STSTL = DSQRT ((SUM2 - SUM**2/DFLOAT(KOUNT))/
     1            DFLOAT(KOUNT - 1))
      COSTL = STSTL/AVSTL
C
C     PRINT CHECK ON DISTRIBUTION OF STEEL RESISTANCES
C
      WRITE (6,2003) STRSTL, COVSTL, AVSTL, COSTL
 2003 FORMAT (1X,// 'TARGET ULTIMATE STEEL
RESISTANCE:',//,
     1        5X, 'AVERAGE ULTIMATE STEEL RESISTANCE
=',F8.3,/,
     2        5X, 'COV OF ULTIMATE STEEL RESISTANCES
=', F8.3///
     3        1X, 'ACTUAL ULTIMATE STEEL
RESISTANCES:',//,
     4        5X, 'AVERAGE ULTIMATE STEEL RESISTANCE
=', F8.3,/,
```

```
     5          5X, 'COV OF ULTIMATE STEEL RESISTANCES
=',F8.3,///)
C
C      SET UP STATISTICAL DISTRIBUTION OF CONCRETE
RESISTANCES.
C      THIS SUBROUTINE RETURNS NMAX VALUES OF CONCRETE
C      RESISTANCE, DISTRIBUTED ACCORDING TO THE ABOVE.
C
      CALL RESCONC
C
C      CHECK DISTRIBUTION OF CONCRETE RESISTANCES
C
      KOUNT = 0
      SUM = 0.D0
      SUM2 = 0.D0
      DO 15 I=1,NMAX
      KOUNT = KOUNT + 1
      SUM = SUM + DISTCON(I)
      SUM2 = SUM2 + DISTCON(I)**2
   15 CONTINUE
C
      AVCONC = SUM/DFLOAT(KOUNT)
      STCONC = DSQRT ((SUM2 - SUM**2/DFLOAT(KOUNT))/
     1             DFLOAT(KOUNT - 1))
      COCONC = STCONC/AVCONC
C
C      PRINT CHECK ON DISTRIBUTION OF CONCRETE
RESISTANCES
C
      WRITE (6,2004) STRCONC, COVCONC, AVCONC, COCONC
 2004 FORMAT (1X,// 'TARGET CONCRETE RESISTANCE:',///,
```

```
     1          5X, 'AVERAGE CONCRETE RESISTANCE =',
F8.3,/,
     2          5X, 'COV OF CONCRETE RESISTANCES
=',F8.3,///,
     3          1X, 'ACTUAL CONCRETE RESISTANCES:',//,
     4          5X, 'AVERAGE CONCRETE RESISTANCE =',
F8.3/,
     5          5X, 'COV OF CONCRETE RESISTANCES =',
F8.3,///)
C
C     APPLY MONTE CARLO TECHNIQUE NMAX TIMES.  EACH
TIME,
C     DETERMINE THE LESSER OF STEEL OR CONCRETE
RESISTANCE.
C     CALCULATE (CONCRETE - STEEL), AND ACCUMULATE THE
C     STATISTICS NECESSARY TO DETERMINE THE MEAN AND
C     STANDARD DEVIATION OF (CONCRETE - STEEL).
C
      KOUNT = 0
      SUM = 0.D0
      SUM2 = 0.D0
C
      DO 20 I=1,NMAX
      DIFF(I) = DISTCON(NRAN(I,2)) - DISTSTL(NRAN(I,2))
C
      KOUNT = KOUNT + 1
      SUM = SUM + DIFF(I)
      SUM2 = SUM2 + DIFF(I)**2
   20 CONTINUE
C
      AVGDIFF = SUM/DFLOAT(KOUNT)
```

```
      STDDIFF = DSQRT ((SUM2 - SUM**2/DFLOAT(KOUNT))/
     1             DFLOAT(KOUNT - 1))
C
C     GIVEN THE STATISTICAL CHARACTERISTICS OF THE
C     (CONCRETE - STEEL) CURVE, COMPUTE THE SAFETY
C     INDEX BETA, AND THE PROBABILITY OF BRITTLE
C     FAILURE.
C
      BETA = AVGDIFF/STDDIFF
      PROB =
AREA(0.D0,AVGDIFF,STDDIFF)/AREA(10.D0,AVGDIFF,STDDIFF)
C
C     WRITE OUTPUT
C
      WRITE (6,2005)
 2005 FORMAT (/,8X,'BETA',10X,'PROBABILITY OF CONCRETE
FAILURE'/)
      WRITE (6,2006) BETA, PROB
 2006 FORMAT (1X,F10.2,15X,E10.3)
      STOP
      END
C*****************************************************
**
      SUBROUTINE RANDM
      IMPLICIT REAL*8 (A-H,O-Z)
      INTEGER*2 IHR, IMIN, ISEC, I100TH
      PARAMETER (NMAX = 10000)
      REAL*4 RANVAL
      COMMON / RAN / NRAN(NMAX,2), DISTLOD(NMAX),
     1             DISTSTL(NMAX), DISTCON(NMAX),
DIFF(NMAX)
```

```
C
C     GENERATE 2 DIFFERENT SETS OF
C     PSEUDO-RANDOM NUMBERS, USING A DIFFERENT SEED
EACH TIME.
C
      DO 20 J=1,2
      CALL GETTIM (IHR,IMIN,ISEC,I100TH)
      CALL SEED(I100TH)
      DO 10 I=1,NMAX
C
C     RANVAL RETURNS A SINGLE RANDOM NUMBER GREATER
THAN OR EQUAL
C     TO ZERO AND LESS THAN 1.0.
C
      CALL RANDOM (RANVAL)
      NRAN(I,J) = INT4(RANVAL*DFLOAT(NMAX))
   10 CONTINUE
   20 CONTINUE
C
      RETURN
      END
C*******************************************************
*******
      SUBROUTINE RESSTL
      IMPLICIT REAL*8 (A-H,O-Z)
      PARAMETER (NMAX = 10000)
      COMMON / STAT / AVGLOD, COVLOD, AVGSTL, COVSTL,
STRSTL,
     1                AVGCONC, COVCONC, STRCONC
      COMMON / RAN / NRAN(NMAX,2), DISTLOD(NMAX),
```

```
     1                DISTSTL(NMAX), DISTCON(NMAX),
DIFF(NMAX)
C
C      SET UP STATISTICAL DISTRIBUTION OF ULTIMATE
RESISTANCE.
C      ULTIMATE RESISTANCE EQUALS DESIGN LOAD TIMES LOAD
FACTOR,
C      DIVIDED BY DIVIDED BY PHI FACTOR, MULTIPLIED BY
RATIO OF
C      SPECIFIED ULTIMATE RESISTANCE TO SPECIFIED YIELD
C      RESISTANCE, AND FINALLY MULTIPLIED BY AVERAGE
RATIO
C      OF ACTUAL TO PREDICTED STEEL CAPACITY.
C
       DESLOD = AVGLOD*(1.D0 + COVLOD*1.924D0)
       PHISTL = 0.9D0
C
C      DETERMINE DESIGN STRENGTH OF STEEL
C      COEFFICIENT OF VARIATION WILL BE
C      AS GIVEN IN DATA STATEMENTS ABOVE.
C      THIS ASSUMES THAT STEEL DESIGN BASED ON PHI*FY
WILL GOVERN
C      OVER STEEL DESIGN BASED ON 0.8*FUT
       STRSTL = (DESLOD*1.7D0/PHISTL)*AVGSTL
C
C      SET UP STATISTICAL DISTRIBUTION OF STEEL
C      RESISTANCES.  COEFFICIENT OF VARIATION
C      DOES NOT CHANGE.
C
       XSTART = STRSTL*(1.D0 - 5.D0*COVSTL)
       XEND = STRSTL*(1.D0 + 5.D0*COVSTL)
```

```
      DX = (XEND - XSTART)/DFLOAT(NMAX/50 + 1)
      KOUNT = 0
      SUM = 0.D0
C
      DO 20 J=1,NMAX/50 + 1
      X = XSTART + DX/2.D0 + DFLOAT(J-1)*DX
      SUM = SUM + GAUSS(X,STRSTL,STRSTL*COVSTL)
   20 CONTINUE
C
      FACTOR = DFLOAT(NMAX)/SUM
      KOUNT = 0
C
      DO 30 J=1,NMAX/50 + 1
      X = XSTART + DX/2.D0 + DFLOAT(J-1)*DX
      NHIST = INT4(FACTOR *
GAUSS(X,STRSTL,STRSTL*COVSTL))
      DO 29 K=1,NHIST
      KOUNT = KOUNT+1
      DISTSTL(KOUNT) = X
   29 CONTINUE
   30 CONTINUE
C
      IF (KOUNT.LT.NMAX) THEN
            DO 35 I=KOUNT+1,NMAX
            DISTSTL(I) = STRSTL
   35       CONTINUE
      ENDIF
C
      RETURN
      END
```

```
C*****************************************************
*********
      SUBROUTINE RESCONC
      IMPLICIT REAL*8 (A-H,O-Z)
      PARAMETER (NMAX = 10000)
      COMMON / STAT / AVGLOD, COVLOD, AVGSTL, COVSTL,
STRSTL,
     1                AVGCONC, COVCONC, STRCONC
      COMMON / RAN / NRAN(NMAX,2), DISTLOD(NMAX),
     1               DISTSTL(NMAX), DISTCON(NMAX),
DIFF(NMAX)
C
C     SET UP STATISTICAL DISTRIBUTION OF CONCRETE
RESISTANCE.
C     DESIGN RESISTANCE EQUALS THE DESIGN STEEL
STRENGTH
C     (DESLOD/PHISTEEL), MULTIPLIED BY AVERAGE RATIO OF
STEEL
C     ULTIMATE STRENGTH TO STEEL YIELD STRENGTH,
DIVIDED
C     BY THE PHI FACTOR FOR CONCRETE, AND MULTIPLIED BY
THE
C     AVERAGE RATIO OF ACTUAL CONCRETE STRENGTH TO
COMPUTED
C     CONCRETE STRENGTH.  THE AVERAGE RATIO OF STEEL
ULTIMATE
C     STRENGTH TO STEEL YIELD STRENGTH IS
CONSERVATIVELY
C     TAKEN AS 1.20.
C
      PHISTL = 0.9D0
```

```
      PHICONC = 0.65D0
C
C     DETERMINE DESIGN STRENGTH OF CONCRETE
C     COEFFICIENT OF VARIATION WILL BE
C     AS GIVEN IN DATA STATEMENTS ABOVE.
C
      DESLOD = AVGLOD*(1.D0 + COVLOD*1.924D0)
      STRCONC =
((DESLOD*1.7D0)/PHISTL)*1.2D0*AVGCONC/PHICONC
C
C     SET UP STATISTICAL DISTRIBUTION OF CONCRETE
C     RESISTANCES.  COEFFICIENT OF VARIATION
C     DOES NOT CHANGE.
C
      XSTART = STRCONC*(1.D0 - 5.D0*COVCONC)
      XEND = STRCONC*(1.D0 + 5.D0*COVCONC)
      DX = (XEND - XSTART)/DFLOAT(NMAX/50 + 1)
      KOUNT = 0
      SUM = 0.D0
C
      DO 20 J=1,NMAX/50 + 1
      X = XSTART + DX/2.D0 + DFLOAT(J-1)*DX
      SUM = SUM + GAUSS(X,STRCONC,STRCONC*COVCONC)
   20 CONTINUE
C
      FACTOR = DFLOAT(NMAX)/SUM
      KOUNT = 0
      SUM = 0.D0
C
      DO 30 J=1,NMAX/50 + 1
      X = XSTART + DX/2.D0 + DFLOAT(J-1)*DX
```

```
      NHIST = INT4(FACTOR *
GAUSS(X,STRCONC,STRCONC*COVCONC))
      DO 29 K=1,NHIST
      KOUNT = KOUNT+1
      DISTCON(KOUNT) = X
   29 CONTINUE
   30 CONTINUE
C
      IF (KOUNT.LT.NMAX) THEN
            DO 35 I=KOUNT+1,NMAX
            DISTCON(I) = STRCONC
   35       CONTINUE
      ENDIF
C
      RETURN
      END
C*******************************************************
*********
      FUNCTION AREA(XEND,AVG,STD)
C
C     THIS FUNCTION SUBPROGRAM COMPUTES THE AREA UNDER
A GAUSSIAN

C     CURVE BETWEEN -10 AND XEND.
C
      IMPLICIT REAL*8 (A-H,O-Z)
C
C     SET UP LIMITS OF NUMERICAL INTEGRATION
      START = -10.D0
      DX   = (XEND - START)/1000.D0
      AREA = 0.D0
```

```
C
      DO 10 N=1,1000
      X1 = START + DFLOAT(N-1)*DX
      X2 = X1 + DX
      DA = DX*(GAUSS(X1,AVG,STD) +
GAUSS(X2,AVG,STD))/2.D0
      AREA = AREA + DA
   10 CONTINUE
C
      RETURN
      END
C*******************************************************
**********
      FUNCTION GAUSS(X,AVG,STD)
      IMPLICIT REAL*8 (A-H,O-Z)
C
C     THIS FUNCTION SUBPROGRAM RETURNS THE VALUE OF A
GAUSSION
CURVE
C     FOR ANY AVERAGE (AVG) AND STANDARD DEVIATION
(STD).
C
      PI = 2.D0*DASIN(1.D0)
      GAUSS = 1.D0/(STD*DSQRT(2.D0*PI))*
     1        DEXP(-((X - AVG)**2)/(2.D0*STD**2))
C
      RETURN
      END
```

APPENDIX K: EFFECT OF DIFFERENT ASSUMED STATISTICAL DISTRIBUTION ON STRENGTHS AND RESISTANCES

K-1 Ratio of Actual to Predicted Capacity Distribution

The LRFD design approach is based on a first and second moment probabilistic methods; that is the first and second moment (mean and standard deviation) of the random variable are used. Many different probability distributions can be used to approximate the capacity of structural members. Among these are the Normal, Log-normal, Weibull, and the Gumbel distributions. In this appendix the ratio of actual concrete capacity to the predicted concrete capacity is tested for the normal and log-normal distribution. The ratio of actual steel capacity to the predicted capacity is tested for the normal, log-normal, Weibull, and Gumbel distributions.

K-1.1.1 Normal Distribution:

It is often assumed that the random variable is normally distributed, and is described in terms of the mean and the standard deviation. Eq. 1 gives the closed format for the probability density function of such a distribution:

$$f(x) = \left(\frac{1}{\sigma\sqrt{2\pi}}\right) e^{\left[\frac{-(x-\mu)^2}{2\sigma^2}\right]} \qquad \textbf{(1)}$$

Here, σ is the standard deviation and μ is the mean of the random variable x.

K-1.1.2 Log-Normal Distribution:

A log-normal distribution is skewed about the mean, and is described in terms of the mean μ and the standard deviation σ. Because this distribution is valid for only positive real numbers, the left tail of the distribution must approach zero faster than the right. The form of the probability density function is given in Eq. 2.

$$f(x) = \left[\frac{1}{x\,\sigma_y\,\sqrt{2\,\pi}}\right] e^{\left[-\frac{1}{2\,\sigma_y^2}\,(\ln(x)\,-\,\mu_y)^2\right]} \qquad \textbf{(2)}$$

where μ_y and σ_y are given by Eq. 3 and Eq. 4 respectively:

$$\mu_y = \ln(\mu) - \frac{1}{2}\ln\left(1+\left(\frac{\sigma}{\mu}\right)^2\right) \qquad \textbf{(3)}$$

$$\sigma_y = \sqrt{\left[\ln\left(1+\left(\frac{\sigma}{\mu}\right)^2\right)\right]} \qquad \textbf{(4)}$$

In the above, σ is the standard deviation and μ is the mean of the random variable x.

K-1.1.3 Weibull Distribution:

The Weibull distribution is an extreme value distribution widely used in reliability models for lifetimes of devices [14]. The distribution is defined in terms of two parameters: the shape factor α and the scale parameter β. These two parameters are obtained from test data. The probability density function is given by Eq. 5:

$$f(x) = \alpha\ \beta^{(-\alpha)}\ x^{(\alpha-1)}\ e^{\left[-\left(\frac{x}{\beta}\right)^{\alpha}\right]} \qquad (5)$$

The shape factor α and the scale factor β must satisfy Eq. 6 and Eq. 7:

$$\frac{\sum x_i^{\alpha} \ln(x_i)}{\sum x_i^{\alpha}} - \frac{1}{\alpha} = \frac{\sum \ln(x_i)}{n} \qquad (6)$$

$$\beta = \left(\frac{\sum x_i^{\alpha}}{n}\right)^{\frac{1}{\alpha}} \qquad (7)$$

The above two equations can be solved numerically by Newton's method for the values of α and β [18].

K-1.1.4 Gumbel Distribution:

This extreme value distribution has two forms one skewed to the right and the other to the left. Only the distribution skewed to the left is used in this appendix. The distribution is defined in terms of a location parameter and a scale parameters. The distribution is defined as in Eq. 8:

$$f(x) = \frac{1}{\sigma} e^{\left[-\frac{1}{\sigma}(x-\mu)-e^{\left(-\frac{1}{\sigma}(x-\mu)\right)}\right]} \qquad \textbf{(8)}$$

the scale parameter is defined by Eq. 9 and the location parameter by Eq. 10

$$\sigma = \frac{1}{1.283}\sigma_x \qquad \textbf{(9)}$$

$$\mu = \mu_x - 0.577\,\sigma \qquad \textbf{(10)}$$

where σ_x is the standard deviation and μ_x is the mean of the random variable x

K-1.2.1 Determination of the Best Probability Distribution to Describe Concrete Capacities:

Using known values of the ratio of actual to predicted capacity of single anchors in tension, governed by concrete failure, for different ranges of embedment depths and for the three theories, the normal and log-normal distributions were tested. The following ranges of embedment depths are considered: 0.01-1.0 in, 1.01-2.0 in, 2.01-3.0 in, 3.01-4.0 in, 4.01-5.0 in, 5.01-6.0 in, 6.01-8.0, and greater than 8.01 in. Although the exact approach would have been to take a discrete embedment depth and determine the probability density function that best fit the actual histogram, this was not possible due to the variability and/or lack of data for certain embedment depths.

Based on the available data for concrete failure,the ratios of actual to predicted capacities are computed. The ratios of actual to predicted concrete capacity are discretized using increments of 0.05. For example ratios of 0.63, 0.64, 0.65, 0.66, and 0.67 are all considered as 0.65. While this discretization may produce some increments with apparent zero probabilities, this is not significant since the actual mean and standard deviation are used in plotting the probability density functions. The actual, normal, and log-normal probability distribution are computed for each histogram.

The error associated with each assumed distribution was evaluated by computing the square root of the sum of the square error between the actual probability of failure and the predicted probability of failure by either the normal or the log-normal distribution, for each range of embedment depths. The results are expressed in Table 1 and shown in the form of bar charts for the ACI 349-85 (Fig. 1), variable-angle cone (Fig. 2), and the Kappa theory (Fig. 3).

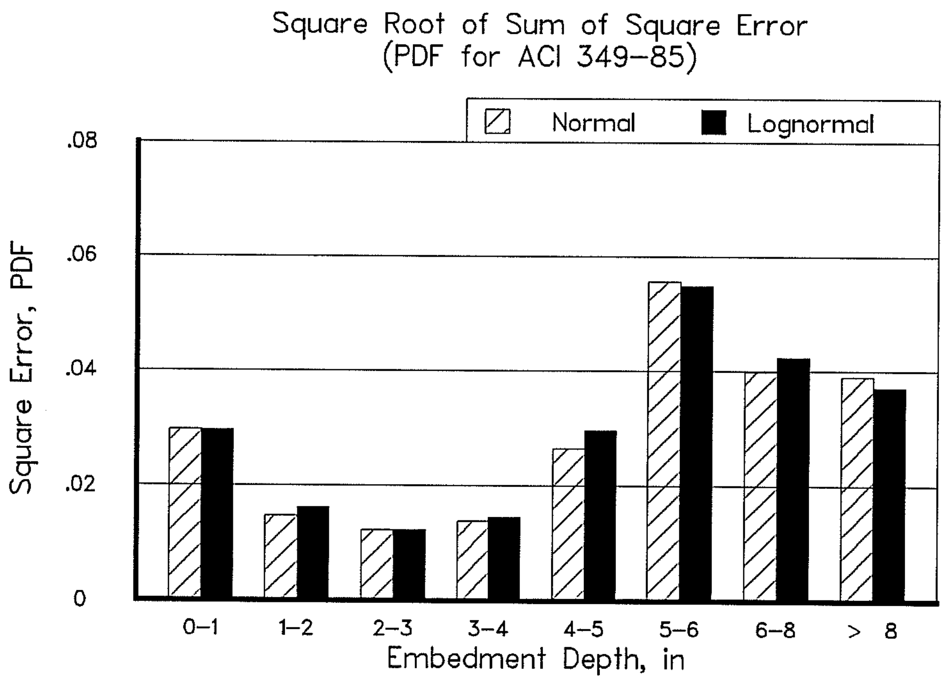

Figure 1: Square Root of Sum of Square Error for ACI 349-85 Concrete Capacity Distributions

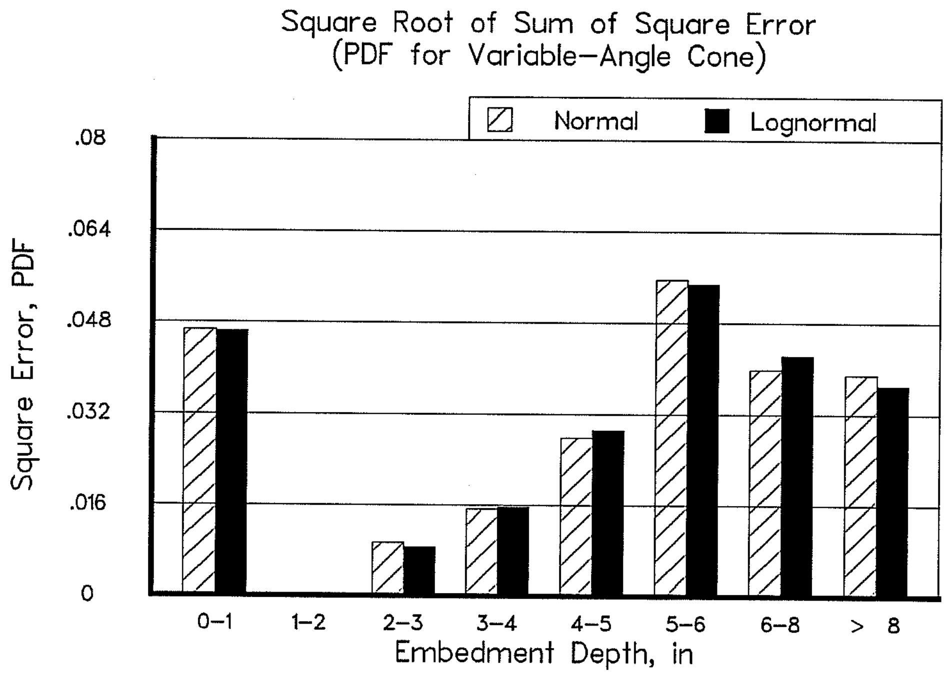

Figure 2: Square Root of Sum of Square Error for Variable-angle Cone Concrete Capacity Distributions

Table 1 and Figs. 1-3 show that the log-normal distribution has a smaller error than the normal distribution for some ranges and a larger error for other ranges. It is clear from Table 1 and Figs. 1-3 that the error associated with each distribution is very small. Hence, either distribution can be used to approximate the ratio of actual to predicted capacity. The normal distribution is used for capacities and also for loads.

Table 1: Square Root of Sum of Square Errors for Concrete Capacity, Using Normal and Log-normal Distributions

Embedment Depth		0.01 - 1.0 in	1.01 - 2.0 in	2.01 - 3.0 in	3.01 - 4.0 in	4.01 - 5.0 in	5.01 - 6.0 in	6.01 - 8.0 in	> 8.01 in
ACI 349-85	Normal	.0297	.0147	.0123	.0138	.0264	.0556	.0398	.0389
	Log-normal	.0295	.0162	.0123	.0145	.0295	.0548	.0423	.037
Variable-angle cone	Normal	.0467	2.93×10^{-4}	.00943	.0153	.0279	.0556	.0398	.0389
	Log-normal	.0464	3.20×10^{-4}	.00862	.0157	.0292	.0548	.0423	.037
Kappa Theory	Normal	.0377	.0124	.0099	.0178	.0228	.0686	.0013	.451
	Log-normal	.0279	.015	.0112	.0163	.0225	.0595	.0014	.042

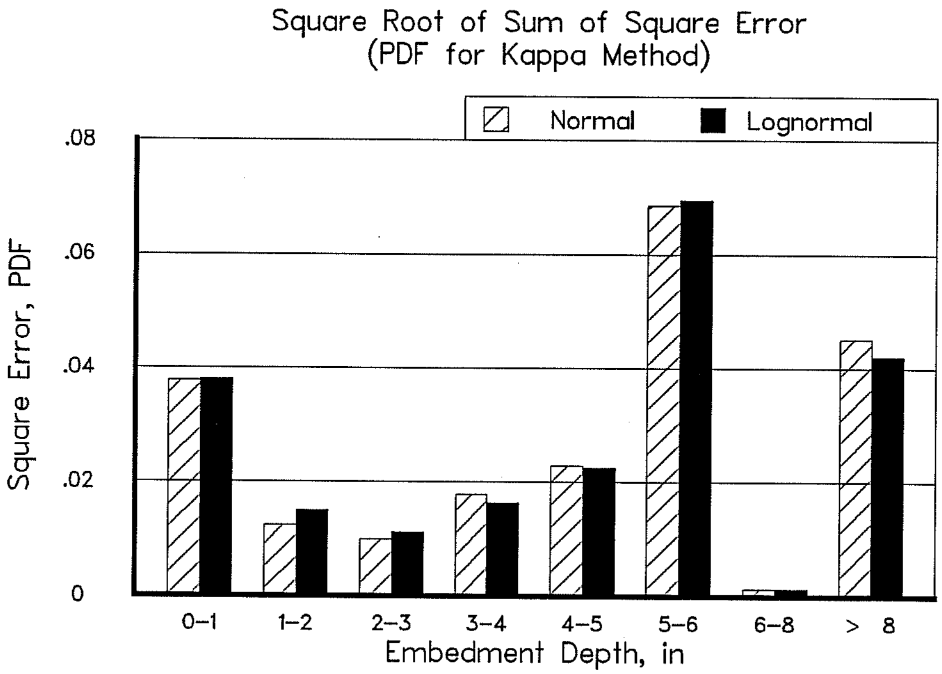

Figure 3: Square Root of Sum of Square Error for Kappa Theory Concrete Capacity Distributions

K-1.2.2 Steel Capacity Distribution:

The available steel data is tested for four distributions: normal, log-normal, Weibull, and Gumbel. The same procedure used with the concrete data is again used here. That is, the mean and standard deviation of the ratios of actual to predicted capacity are computed. The ratios are discretized using intervals of 0.05. The mean and standard deviation of the resulting histogram are then used to compute the probability density function corresponding to each distribution. The steel data is divided into two parts. One consists of high strength steel for which f_{ut} is 125 ksi (A193-B7); the other consist of low strength steel for which f_{ut} is 60 ksi (A307 and stud). The probability density function with the smallest sum of square error is considered to best represent the data. The sum of the square error is computed for data at points located at a mean one minus standard deviation, and at the mean plus a standard deviation.

The best distribution is picked based on the sum of the square root of the sum square errors. Table 2 presents the sum of square error for high strength steel anchors.

Table 2: Square Root of Sum of Square Errors for Different Assumed Distribution

Distribution	Error
Normal	0.0688
Log-Normal	0.0659
Weibull	0.0717
Gumbel	0.0621

Table 2 shows that any of the four distributions can be used. The sum of square error is about the same for the normal, log-normal, Gumbel, Weibull distribution. The normal distribution is used for steel capacity.

REFERENCES

1. ACI Committee 349, Code Requirements for Nuclear Safety Related Structures (ACI 349-85), American Concrete Institute, Detroit, 1985.

2. Eligehausen, R., Fuchs, W., and Breen, J. E., "Fastening to Concrete Design of Fastenings Using Steel Anchors or Headed Studs Comparison of Design Procedures, Report No. 12/14 - 91/10 to ACI 318 SubCommittee B," The University of Texas at Austin, 1991.

3. Cannon, R. w., Personal communication, December 2, 1991

4. Eligehausen, R., Personal communication,December 23, 1991

5. CEB Task Group VI/5, Fastenings to Reinforced Concrete and Masonry Structures: Design and Detailing; Volume 1 (Behavior under Monotonic, Sustained, Fatigue, Seismic and Impact Loadings, Task Group/5 (Embedments), Euro-International Concrete Committee (CEB), May 1991.

6. Eligehausen, R., Fuchs, W., and Mayer, B., "Tragverhalten von Dubelbefestigungen bei

Zugbeanspruchung" ("Load-bearing Behavior of Anchor Fastenings in Tension"), Betonwerk + Fertigteil-Technik, Berlin, no. 12 (1987). pp. 826-832 (German) and no. 1 (1988), pp. 29-35 (English), 1987/88.

7. Rehm, G., Eligehausen, R., and Mallee, R., "Befestigungstechnik," Beitrag zum Betonkalendar 1988 ("Fastening Technique," contribution to 1988 Concrete Calendar), Verlag Enst & Sohn, Berlin, 1988.

8. Klingner, R. E. and Mendonca, J. A., "Tensile Capacity of Short Anchor Bolts and Welded Studs: A Literature Review," Journal of the American Concrete Institute, Proceedings Vol. 79, No. 4, July-August 1982, pp. 270-279.

9. Collins, D. M., Klingner, R. E., and Polyzois, D., "Load-Deflection Behavior of Cast-in-Place and Retrofit Concrete Anchors Subjected to Static, Fatigue, and Impact Tensile Loads," Research Report CTR 1126-1, Center for Transportation Research, The University of Texas at Austin, February 1989.

10. Eligehausen, R. and Sawade, G., "A Fracture Mechanics-Based Description of the Pull-Out Behavior of Headed Studs Embedded in Concrete," University of Stuttgart, 1989. (Especially see Fig. 10).

11. "General Anchorage to Concrete," TVA Civil Design Standard No. DS-C.1.7.1, Tennessee Valley Authority, Knoxville, Tennessee, 1984.

12. Eligehausen, R., "Report to ACI 355," 1989.

13. Avram, Constantin, et al, Concrete Strength and Strains, Elsevier Scientific Publishing Company, Oxford, England, 1981.

14. Lee, D. W., and Breen, J. E., "Factors Affecting Anchor Bolt Development," Research Report No. 88-1F, Center for Highway Research, The University of Texas at Austin, Aug. 1966.

15. Hasselwander, G. B., Jirsa, J. O., Breen, J. E., and Lo, K., "Strength and Behavior of Anchor Bolts Embedded near Edges of Concrete Piers," Research Report No. 29-2F, Center for Highway Research, The University of Texas at Austin, May 1977.

16. Ellingwood, B., et al., "Development of a Probability Based Load Criterion for American National Standard A58 (Building Code Requirements for Minimum Design Loads in Buildings and Other Structures)," NBS Special Publication 577, U.S. Department of Commerce / National Bureau of Standards, June 1980.

17. Klingner, R. E., "Development of a Load and Resistance Factor Basis for Anchorage Design by ACI

349-85, Appendix B, Report to ACI Committee 355," September 28, 1990

18. MacGregor, J. G., Reinforced Concrete Mechanics and Design, Prentice-Hall, Englewood Cliffs, New Jersey, 1988.

19. Law, A.M., Kelton, W.D., Simulation Modeling and Analysis, McGraw-Hill, New York, 1982.

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