Fermilab

Particle Physics Division
Mechanical Department Engineering Note

Number:	MD-ENG-03-015
Date:	August 5, 2003
Project Reference:	Install and Level Zero Layer, WBS 1.1.2.3.3.3.16, UID 74645
Project:	NuMI
Title:	Zero Layer Grout
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Reviewer(s):	Bob Woods
Key Words:	target pile, target pile shielding, zero layer, grout
Abstract Summary:	The target pile is built on four grouted, steel rails. The grouted rails are shown as small crosshatched areas under the target pile on the sketch in Appendix B. Cooling air flows through the passages formed between the concrete floor, grouted rails, and the target pile. Grout design bearing strength is calculated and compared to the load applied to the steel rails.
Applicable Codes:	Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (ACI 318R-02)

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Target Pile Zero layer grout	AMS	
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1.0 The purpose of this engineering note is to calculate the grout design bearing strength and compare it to the applied load.

2.0 Grout ultimate compressive stress is given in Appendix A. The grout will be installed in the flowable state. However, use f_c' for the fluid state, which is lower, in case too much water is added. $\therefore f_c' = 9{,}000$ psi.

3.0 The applied load is the target pile shielding steel. The steel is shown in Appendix B. The steel is cross hatched with solid lines. The two open areas at the top of the pile, where the carriage support beams run, are treated as if they are filled with steel. The central chase is open area. The grout rails are the four, small, darker cross hatched areas under the steel.

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- Dimensions of the target pile shielding steel:

Use the maximum Duratek block size for this calculation.

52.75" x 52.4 x 26.34"

Pile width = 3 (52.75) = 158.25"

Pile height = 3(52.75) + 26.34 = 184.59"

Aperture dimensions = 52.89" x 52.75"
(158.25 - 4(26.34))

Steel cross sectional area = (158.25)(184.59) - 52.89(52.75)

= 29,211.368 - 2,789.948

= 26,421.42 in^2

= 183.4821 ft^2

Load per foot = $183.48\,\cancel{ft^2} * 1\,\cancel{ft} * 490\,\frac{Lbs}{\cancel{ft^3}} = 89,906\,\frac{Lbs}{ft}$

$= 44.953\,\frac{tons}{ft}$ ✓

- Quick check

$19\,\cancel{blocks} * 10.327\,\frac{tons}{\cancel{block}} = 196.131$ tons per 52.4"

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= 44.915 tons/ft ✓

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Duratek Block WT = $\frac{52.75 \times 52.4 \times 26.34}{1728} \times \frac{490}{2000}$

= 10.323 tons/block

- Load distribution on the rails

Load on an outer rail & the inner rail adjacent to it:

Steel cross sectional area = $\overset{\text{PILE HT}}{184.59}(2 \times 26.34)$

= 9,724 in² = 67.53 ft²

Load per foot = 67.53 $\cancel{ft^2} \times 1 \cancel{ft} \times 490 \frac{Lbs}{\cancel{ft^3}}$ = 33,089 $\frac{Lbs}{ft}$

16.545 $\frac{tons}{ft}$

Load per rail = $\frac{16.545}{2}$ = 8.272 tons/ft

Additional load on the inner rails:

Width = 158.25 − 4(26.34) = 52.89"

Height = 184.59 − 52.75 = 131.84"

Steel cross sectional area = 52.89(131.84) = 6973 in²

= 48.424 ft²

Load per foot = 48.424 ft² × 1 ft × 490 $\frac{Lbs}{ft^3}$ = 23,728 $\frac{Lbs}{ft}$ = 11.864 $\frac{ton}{ft}$

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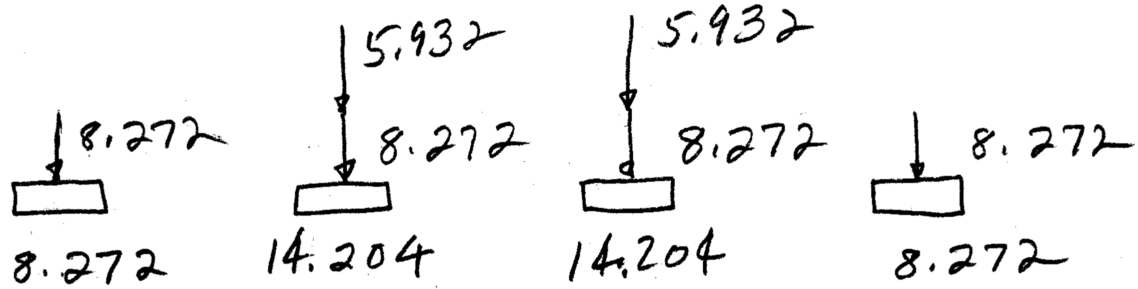
Additional Load per inner rail $= \frac{11.864}{2} = 5.932 \frac{\text{tons}}{\text{ft}}$

Load summary:



Total wt/ft $= 2(8.272 + 14.204) = 44.952 \frac{\text{tons}}{\text{ft}}$ ✓

$\therefore$ The heaviest loaded rail is $14.204(1.1) = 15.6$

Use 16 tons/ft.

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Grout loading

Total weight along one block length

$= 16 \frac{\cancel{tons}}{\cancel{ft}} * 2{,}000 \frac{Lbs}{\cancel{ton}} * 52.75 \cancel{in} * \frac{\cancel{ft}}{12 \cancel{in}}$

$= 140{,}667$ Lbs (70.3 tons)

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Assume the bottom block rests on 2 points.

∴ Maximum reaction force = 140,667 Lbs / 2

= 70,334 Lbs

Grout design bearing strength Refer to Appendix C.

$= 0.85 \phi f_c' A_1 \sqrt{\frac{A_2}{A_1}} \rightarrow$ Limit $\sqrt{\frac{A_2}{A_1}}$ to 2 max.

The load spreads out along a 2:1 angle according to the calculation procedure in Appendix A. The Duratek blocks rest on top of a 2" thick steel bar that sits on top of the 2" thick layer of grout.

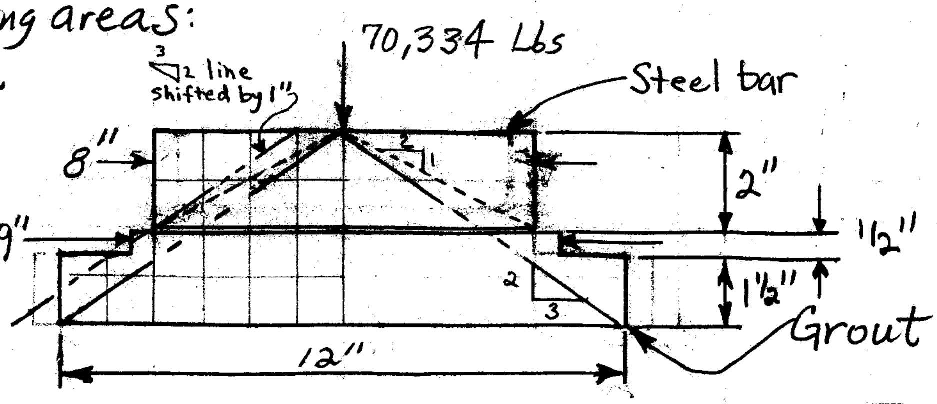
Circular bearing areas:

$A_1 = \pi\left(\frac{2}{1} \times 2\right)^2$

$= 50.3 \text{ in}^2$

$A_2 = \pi\left(\frac{9}{2}\right)^2$

$= 63.6 \text{ in}^2$



$$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{63.6}{50.3}} = 1.12$$

$$f'_c = 9{,}000 \text{ psi}$$

$$\phi = 0.65$$

$\therefore$ Grout design bearing strength

$= 0.85(0.65)(9{,}000 \text{ psi})(50.3 \text{ in}^2)(1.12)$

$= 280{,}130$ Lbs

Factored load $= 1.4(70{,}334 \text{ Lbs}) = 98{,}500$ Lbs

$< 280{,}130$ Lbs $\underline{\underline{\text{OK}}}$

$280{,}130/98{,}500 = 2.8$

The load might spread out along a $\overset{3}{\triangleleft}{}_{2}$ angle as shown in the sketch on page 4.

Then: $A_1 = \pi\left(\frac{3}{2} \times 2\right)^2 = 28.3 \text{ in}^2$

$A_2 = \pi\left(\frac{3}{2} \times 4\right)^2 = 113 \text{ in}^2$

$$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{113}{28.3}} = 2$$

$\therefore$ Grout design bearing strength

$= 0.85(0.65)(9{,}000 \text{ psi})(28.3 \text{ in}^2)(2)$

$= 281{,}440 \text{ Lbs} > 98{,}500 \text{ Lbs}$ $\underline{\underline{\text{OK}}}$ $\quad \frac{281{,}440}{98{,}500} = 2.8$

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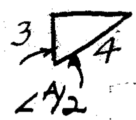
Check grout with load offset by 1".

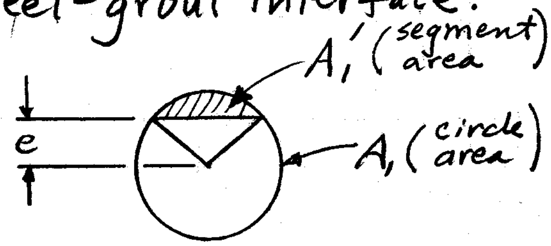
Consider 2:1 angle.

$\bar{A}_1$ is reduced bearing area at steel-grout interface.

$r = \frac{2}{1}(2) = 4''$ $h = 1$

$e = r - h = 4 - 1 = 3''$





$\cos\left(\frac{A}{2}\right) = 3/4$

$A/2 = 41.41°$

$A = 82.82° = 1.445$

$A_1' = \frac{1}{2}(4)^2 in^2 (1.445 - \sin 82.82°)$

$= 3.6\ in^2$

Segment (Fig. 2.1.36) Area = ½r^2(rad A − sin A) = ½[$r(s - c) + ch$], where rad A radian measure of angle A. For small arcs, s = ⅓(8c' − c), where c' = chord of half of the arc (Huygen's approximation).

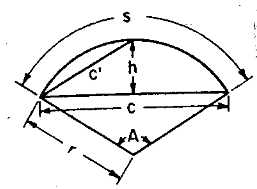


Fig. 2.1.36 Segment.

$\bar{A}_1 = A_1 - A_1' = 50.3 - 3.6 = 46.7\ in^2 = \bar{A}_1$

Don't include $\sqrt{A_2'/A_1'}$. (Since design is ok without including it.)

Grout design bearing strength

$= 0.85(0.65)(9{,}000\ psi)(46.7\ in^2)$

$= 232{,}215\ Lbs > 98{,}500\ Lbs$ <u>OK</u>

$\frac{232{,}215}{98{,}500} = 2.3$

Consider $\overset{3}{\triangleleft}2$ angle.

A_1 is not reduced. $A_1 = 28.3\ in^2$

A_2 is not reduced. $A_2 = 113\ in^2$ This step assumes that the sharp corner in the grout, where the diameter changes from 9" to 12", does not affect the calculation.

∴ Grout bearing design strength

= 281,440 Lbs > 98,500 Lbs, same as on pg 5

Assume the 1" offset load spreads out at an angle which just misses the sharp corner discussed above.

$$\text{slope} = \frac{\Delta\ \text{vertical}}{\Delta\ \text{horizontal}} = \frac{2.5}{3.5} = \frac{5}{7}$$

$$A_1 = \pi\left(\frac{7}{5}\times 2\right)^2 = 24.6\ in^2$$

$$A_2 = \pi(6-1)^2 = 78.5\ in^2$$

$$\sqrt{\frac{A_2}{A_1}} = \sqrt{\frac{78.5}{24.6}} = 1.8$$

Grout bearing design strength

$= 0.85(0.65)(9{,}000\ psi)(24.6\ in^2)(1.8) = 220{,}180$ Lbs

> 98,500 Lbs <u>OK</u>

$$\frac{220{,}180}{98{,}500} = 2.2$$

Conclusions: 1) For 1 block-length of the target pile the grout design bearing strength is greater than the applied load by a factor of at least 2.

2) The factor of 2 means that the grout rail is ok if the loads from two adjacent block-lengths of the target pile overlap.

3) There is a safety factor of 10,000/9,000 = 1.11 on the grout design bearing strength because I used the compressive strength for fluid consistency instead of the flowable consistency that will be specified.

CRYSTEX®

Appendix A

High Performance, Precision Grade, Non-shrink Structural Grout

Product Description:

CRYSTEX is a long work time, ready-mixed, high strength, highly fluid, controlled expansive grout with superior dynamic load stability.

CRYSTEX contains a balanced blend of washed and graded aggregates, portland cement, plasticizing agents, and a proprietary shrinkage compensating system. This unique shrinkage compensating system in **CRYSTEX** guarantees controlled positive expansion in all directions, developing the densest non-ferrous grout on the market today. The positive expansion of **CRYSTEX** remains constant throughout the life of the grout. **CRYSTEX** is free of iron aggregates, gypsum, carbon, chlorides and corrosive-type materials. It is scientifically proportioned and ready for use at any consistency from plastic to fluid.

Unique Properties:

CRYSTEX provides unique placement properties of extended flow and long work time to the installer. This feature, of over 60 minutes of work time, facilitates placement of a superior performing grout in the most unfavorable of placement conditions. Extended placement time is important in many cases, including, high temperature applications, difficult to reach cavities, installations by grout pump, base plates with lateral shear keys, and extremely large grouting cavities that require continuous placement of over one hour. **CRYSTEX** is the original long work time grout.

Basic Use:

CRYSTEX is used where precision, non-shrink, high-strength, structural grout is required, such as: machinery bases, crane rails, pump and equipment bases in power plants, steel and paper mills, sewage treatment plants; keyways; bed plates where heavy repetitive loading occurs; anchor bolts and dowels; structural steel columns; bearing plates; load bearing masonry walls, light poles and highway signs.

Features and Benefits:

- *Precision-grade, non-shrink.*
- *Long-work time.*
- *High strength.*
- *High fluidity.*
- *Easy placement.*
- *Dynamic load stability.*

Limitations:

CRYSTEX is cement based. Follow ACI recommended practices:

ACI-305 - For hot weather concreting.
ACI-306 - For cold weather concreting.

DO NOT ADD PLASTICIZERS, ACCELERATORS OR ADDITIONAL CEMENT TO CRYSTEX.

Avoid mixing more **CRYSTEX** than can be placed in 60 minutes. Avoid **CRYSTEX** placement when temperatures are, or will be, below 7°C (45°F) within 24 hours. When grouting in depth over 75mm (3 inches) extend **CRYSTEX** with 10mm (3/8") pea gravel. Avoid excessive vibration of foundation or base plate at time of placement due to nearby equipment. Do not over vibrate fluid consistency grout. Not recommended for dry pack applications.

Estimating:

CRYSTEX is available in 25-kilogram (55-pound) bags, which yield 14 liters (0.5 cubic foot) of fluid consistency grout.

Applicable Standards:
CRD C 621
ASTM C 1107

Technical Data:

Approximate Water Requirements
Plastic 3.8 L (4.0 qts.)
Fluid 4.5 L (4.75 qts.)
Working Time: *approximately* 60 minutes
Initial Setting Time: *approximately* 5 hours
Typical Vertical Expansion:

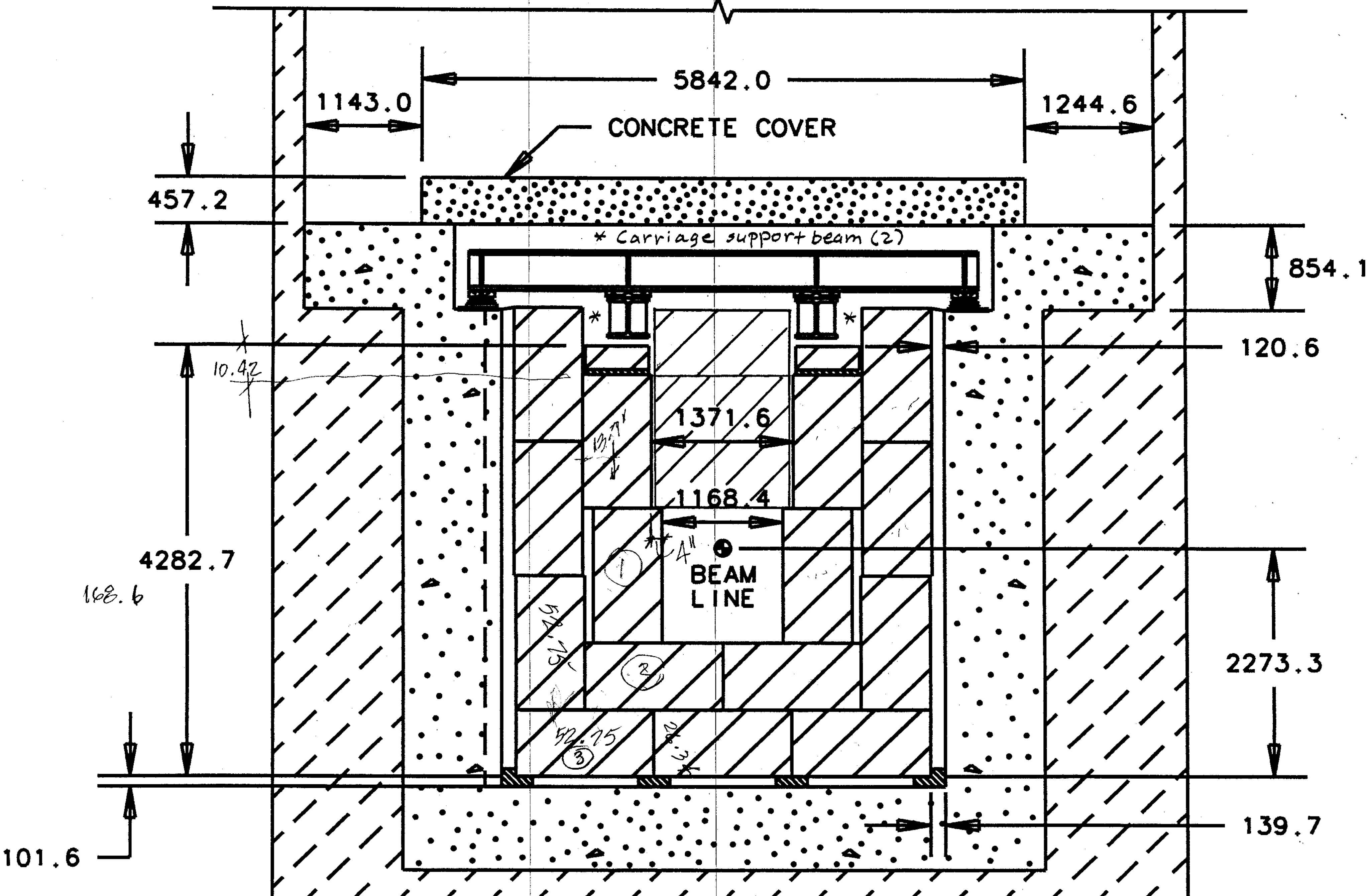
ASTM C 1090		***28 days***
Plastic	100% flow	+0.03%
Fluid	25 sec. flow	+0.02%

Typical Compressive Strength: P.S.I. (Mpa)
ASTM C 1107 and CRD C 621

	1 Day	3 Days	7 Days	28 Days
Plastic	5300 (37)	7200 (50)	8760 (60)	10600 (73)
Flowable	4600 (32)	6460 (45)	8160 (56)	10150 (70)
Fluid	3800 (26)	5700 (39)	7650 (53)	9000 (64)

Appendix B

AMS
8/5/2003



CODE

Appendix C

Source:
Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (ACI 318R-02)

10.17 — Bearing strength

10.17.1 — Design bearing strength of concrete shall not exceed $\phi\,(0.85f_c'A_1)$, except when the supporting surface is wider on all sides than the loaded area, then the design bearing strength of the loaded area shall be permitted to be multiplied by $\sqrt{A_2/A_1}$ but not more than 2.

COMMENTARY

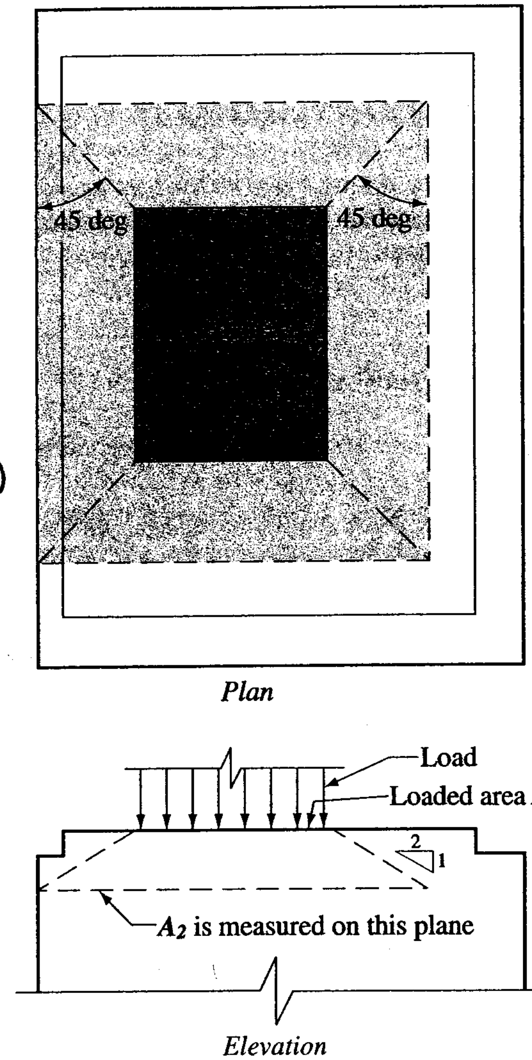


Fig. R10.17—Application of frustum to find A_2 in stepped or sloped supports

R10.17 — Bearing strength

R10.17.1 — This section deals with bearing strength of concrete supports. The permissible bearing stress of $0.85f_c'$ is based on tests reported in Reference 10.41. (See also 15.8).

When the supporting area is wider than the loaded area on all sides, the surrounding concrete confines the bearing area, resulting in an increase in bearing strength. No minimum depth is given for a supporting member. The minimum depth of support will be controlled by the shear requirements of 11.12.

When the top of the support is sloped or stepped, advantage may still be taken of the condition that the supporting member is larger than the loaded area, provided the supporting member does not slope at too great an angle. Fig. R10.17 illustrates the application of the frustum to find A_2. The frustum should not be confused with the path by which a load spreads out as it travels downward through the support.

CODE

9.1.2 — Members also shall meet all other requirements of this code to ensure adequate performance at service load levels.

9.1.3 — Design of structures and structural members using the load factor combinations and strength reduction factors of Appendix C shall be permitted. Use of load factor combinations from this chapter in conjunction with strength reduction factors of Appendix C shall not be permitted.

COMMENTARY

The changes were made to further unify the design profession on one set of load factors and combinations, and to facilitate the proportioning of concrete building structures that include members of materials other than concrete. When used with the strength reduction factors in 9.3, the designs for gravity loads will be comparable to those obtained using the strength reduction and load factors of the 1999 and earlier codes. For combinations with lateral loads, some designs will be different, but the results of either set of load factors are considered acceptable.

Chapter 9 defines the basic strength and serviceability conditions for proportioning structural concrete members.

The basic requirement for strength design may be expressed as follows:

$$\text{Design Strength} \geq \text{Required Strength}$$

$$\phi\,(\text{Nominal Strength}) \geq U$$

In the strength design procedure, the margin of safety is provided by multiplying the service load by a load factor and the nominal strength by a strength reduction factor.

9.2 — Required strength

9.2.1 — Required strength U shall be at least equal to the effects of factored loads in Eq. (9-1) through (9-7). The effect of one or more loads not acting simultaneously shall be investigated.

$$U = 1.4(D + \cancel{F}) \qquad (9\text{-}1)$$

$$U = 1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R) \qquad (9\text{-}2)$$

$$U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.8W) \qquad (9\text{-}3)$$

$$U = 1.2D + 1.6W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R) \qquad (9\text{-}4)$$

$$U = 1.2D + 1.0E + 1.0L + 0.2S \qquad (9\text{-}5)$$

$$U = 0.9D + 1.6W + 1.6H \qquad (9\text{-}6)$$

$$U = 0.9D + 1.0E + 1.6H \qquad (9\text{-}7)$$

except as follows:

(a) The load factor on L in Eq. (9-3) to (9-5) shall be permitted to be reduced to 0.5 except for garages, areas occupied as places of public assembly, and all areas where the live load L is greater than 100 lb/ft^2.

R9.2 — Required strength

The required strength U is expressed in terms of factored loads, or related internal moments and forces. Factored loads are the loads specified in the general building code multiplied by appropriate load factors.

The factor assigned to each load is influenced by the degree of accuracy to which the load effect usually can be calculated and the variation that might be expected in the load during the lifetime of the structure. Dead loads, because they are more accurately determined and less variable, are assigned a lower load factor than live loads. Load factors also account for variability in the structural analysis used to compute moments and shears.

The code gives load factors for specific combinations of loads. In assigning factors to combinations of loading, some consideration is given to the probability of simultaneous occurrence. While most of the usual combinations of loadings are included, the designer should not assume that all cases are covered.

Due regard is to be given to sign in determining U for combinations of loadings, as one type of loading may produce effects of opposite sense to that produced by another type. The load combinations with **0.9*D*** are specifically included for the case where a higher dead load reduces the effects of other loads. The loading case may also be critical for tension-controlled column sections. In such a case, a reduction in axial load and an increase in moment may result in a critical load combination.

CODE

(b) Where wind load W has not been reduced by a directionality factor, it shall be permitted to use $1.3W$ in place of $1.6W$ in Eq. (9-4) and (9-6).

(c) Where earthquake load E is based on service-level seismic forces, $1.4E$ shall be used in place of $1.0E$ in Eq. (9-5) and (9-7).

(d) The load factor on H shall be set equal to zero in Eq. (9-6) and (9-7) if the structural action due to H counteracts that due to W or E. Where lateral earth pressure provides resistance to structural actions from other forces, it shall not be included in H but shall be included in the design resistance.

9.2.2 — If resistance to impact effects is taken into account in design, such effects shall be included with live load L.

9.2.3 — Estimations of differential settlement, creep, shrinkage, expansion of shrinkage-compensating concrete, or temperature change shall be based on a realistic assessment of such effects occurring in service.

9.2.4 — For a structure in a flood zone, the flood load and load combinations of ASCE 7 shall be used.

9.2.5 — For post-tensioned anchorage zone design, a load factor of 1.2 shall be applied to the maximum prestressing steel jacking force.

COMMENTARY

Consideration should be given to various combinations of loading to determine the most critical design condition. This is particularly true when strength is dependent on more than one load effect, such as strength for combined flexure and axial load or shear strength in members with axial load.

If special circumstances require greater reliance on the strength of particular members than encountered in usual practice, some reduction in the stipulated strength reduction factors ϕ or increase in the stipulated load factors U may be appropriate for such members.

The wind load equation in ASCE 7-98[9.1] and IBC 2000[9.2] includes a factor for wind directionality that is equal to 0.85 for buildings. The corresponding load factor for wind in the load combination equations was increased accordingly (1.3/0.85 = 1.53 rounded up to 1.6). The code allows use of the previous wind load factor of 1.3 when the design wind load is obtained from other sources that do not include the wind directionality factor.

Model building codes and design load references have converted earthquake forces to strength level, and reduced the earthquake load factor to 1.0 (ASCE 7-93[9.3]; BOCA/NBC 93[9.4]; SBC 94[9.5]; UBC 97[9.6]; and IBC 2000[9.2]). The code requires use of the previous load factor for earthquake loads, approximately 1.4, when service-level earthquake forces from earlier editions of these references are used.

R9.2.2 — If the live load is applied rapidly, as may be the case for parking structures, loading docks, warehouse floors, elevator shafts, etc., impact effects should be considered. In all equations, substitute (L + impact) for L when impact should be considered.

R9.2.3 — The designer should consider the effects of differential settlement, creep, shrinkage, temperature, and shrinkage-compensating concrete. The term realistic assessment is used to indicate that the most probable values rather than the upper bound values of the variables should be used.

R9.2.4 — Areas subject to flooding are defined by flood hazard maps, usually maintained by local governmental jurisdictions.

R9.2.5 — The load factor of 1.2 applied to the maximum tendon jacking force results in a design load of about 113 percent of the specified prestressing steel yield strength but not more than 96 percent of the nominal ultimate strength of the prestressing steel. This compares well with the maximum attainable jacking force, which is limited by the anchor efficiency factor.

CODE

9.3 — Design strength

9.3.1 — Design strength provided by a member, its connections to other members, and its cross sections, in terms of flexure, axial load, shear, and torsion, shall be taken as the nominal strength calculated in accordance with requirements and assumptions of this code, multiplied by the strength reduction factors ϕ in 9.3.2, 9.3.4, and 9.3.5.

9.3.2 — Strength reduction factor ϕ shall be as follows:

9.3.2.1 — Tension-controlled sections as defined in 10.3.4 .... 0.90 (See also 9.3.2.7)

9.3.2.2 — Compression-controlled sections, as defined in 10.3.3:

(a) Members with spiral reinforcement conforming to 10.9.3 .... 0.70

(b) Other reinforced members 0.65

For sections in which the net tensile strain in the extreme tension steel at nominal strength is between the limits for compression-controlled and tension-controlled sections, ϕ shall be permitted to be linearly increased from that for compression-controlled sections to 0.90 as the net tensile strain in the extreme tension steel at nominal strength increases from the compression-controlled strain limit to 0.005.

Alternatively, when Appendix B is used, for members in which f_y does not exceed 60,000 psi, with symmetric reinforcement, and with $(h - d' - d_s)/h$ not less than 0.70, ϕ shall be permitted to be increased linearly to 0.90 as ϕP_n decreases from $0.10f_c'A_g$ to zero. For

COMMENTARY

R9.3 — Design strength

R9.3.1 — The design strength of a member refers to the nominal strength calculated in accordance with the requirements stipulated in this code multiplied by a strength reduction factor ϕ, which is always less than one.

The purposes of the strength reduction factor ϕ are (1) to allow for the probability of understrength members due to variations in material strengths and dimensions, (2) to allow for inaccuracies in the design equations, (3) to reflect the degree of ductility and required reliability of the member under the load effects being considered, and (4) to reflect the importance of the member in the structure.[9.7,9.8]

In the 2002 code, the strength reduction factors were adjusted to be compatible with the ASCE 7-98[9.1] load combinations, which were the basis for the required factored load combinations in model building codes at that time. These factors are essentially the same as those published in Appendix C of the 1995 edition, except the factor for flexure/tension controlled limits is increased from 0.80 to 0.90. This change is based on past[9.7] and current reliability analyses,[9.9] statistical study of material properties, as well as the opinion of the committee that the historical performance of concrete structures supports $\phi = 0.90$.

R9.3.2.1 — In applying 9.3.2.1 and 9.3.2.2, the axial tensions and compressions to be considered are those caused by external forces. Effects of prestressing forces are not included.

R9.3.2.2 — Before the 2002 edition, the code specified the magnitude of the ϕ-factor for cases of axial load or flexure, or both, in terms of the type of loading. For these cases, the ϕ-factor is now determined by the strain conditions at a cross section, at nominal strength.

A lower ϕ-factor is used for compression-controlled sections than is used for tension-controlled sections because compression-controlled sections have less ductility, are more sensitive to variations in concrete strength, and generally occur in members that support larger loaded areas than members with tension-controlled sections. Members with spiral reinforcement are assigned a higher ϕ than tied columns since they have greater ductility or toughness.

For sections subjected to axial load with flexure, design strengths are determined by multiplying both P_n and M_n by the appropriate single value of ϕ. Compression-controlled and tension-controlled sections are defined in 10.3.3 and 10.3.4 as those that have net tensile strain in the extreme tension steel at nominal strength less than or equal to the compression-controlled strain limit, and equal to or greater than 0.005, respectively. For sections with net tensile strain

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COMMENTARY

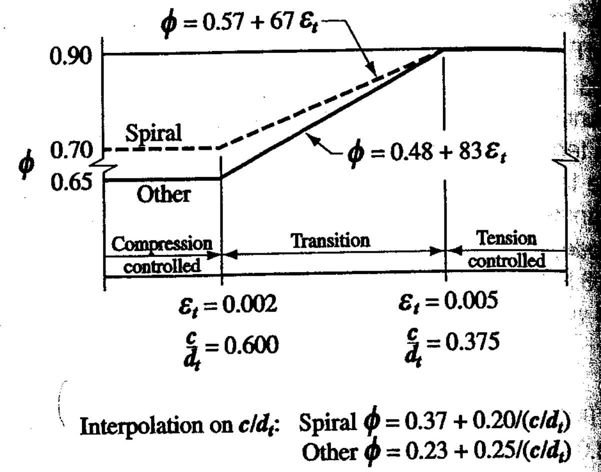

Fig. R9.3.2—Variation of ϕ with net tensile ε_t and c/d_t for Grade 60 reinforcement and for prestressing steel.

other reinforced members, ϕ shall be permitted to be increased linearly to 0.90 as ϕP_n decreases from $0.10f_c'A_g$ or ϕP_b, whichever is smaller, to zero.

ε_t in the extreme tension steel at nominal strength between the above limits, the value of ϕ may be determined by linear interpolation, as shown in Fig. R9.3.2. The concept of net tensile strain ε_t is discussed in R10.3.3.

Since the compressive strain in the concrete at nominal strength is assumed in 10.2.3 to be 0.003, the net tensile strain limits for compression-controlled members may also be stated in terms of the ratio c/d_t, where c is the depth of the neutral axis at nominal strength, and d_t is the distance from the extreme compression fiber to the extreme tension steel. The c/d_t limits for compression-controlled and tension-controlled sections are 0.6 and 0.375, respectively. The 0.6 limit applies to sections reinforced with Grade 60 steel and to prestressed sections. Fig. R9.3.2 also gives equations for ϕ as a function of c/d_t.

The net tensile strain limit for tension-controlled sections may also be stated in terms of the ρ/ρ_b as defined in the 1999 and earlier editions of the code. The net tensile strain limit of 0.005 corresponds to a ρ/ρ_b ratio of 0.63 for rectangular sections with Grade 60 reinforcement. For a comparison of these provisions with the 1999 code Section 9.3, see Reference 9.10.

9.3.2.3 — Shear and torsion 0.75

9.3.2.4 — Bearing on concrete (except for post-tensioned anchorage zones and strut-and-tie models) 0.65

9.3.2.5 — Post-tensioned anchorage zones 0.85

R9.3.2.5 — The ϕ-factor of 0.85 reflects the wide scatter of results of experimental anchorage zone studies. Since 18.13.4.2 limits the nominal compressive strength of unconfined concrete in the general zone to $0.7\lambda f_{ci}'$, the effective design strength for unconfined concrete is $0.85 \times 0.7\lambda f_{ci}' \approx 0.6\lambda f_{ci}'$.

9.3.2.6 — Strut-and-tie models (Appendix A), and struts, ties, nodal zones, and bearing areas in such models 0.75

R9.3.2.6 — The ϕ factor used in strut-and-tie models is taken equal to the ϕ factor for shear. The value of ϕ for strut-and-tie models is applied to struts, ties, and bearing areas in such models.