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Update

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DESIGN OF SLAB-ON-GROUND FOUNDATIONS
An Update

A Design, Construction & Inspection Aid
For Consulting Engineers

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Prepared for:

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INTRODUCTION

In 1981 "DESIGN OF SLAB-ON-GROUND FOUNDATIONS, A Design, Construction & Inspection Aid for Consulting Engineers" was first published. The design procedure set forth in that publication had at that time been in use by the author for about 15 years. After this publication, it was subsequently adopted by the Uniform Building Code (UBC) as Standard 29-4(I). Copies of this work have been distributed by WRI for 22 years to consultants all across the nation. Feedback has been most favorable with no comments of design inadequacy. In a few cases there have been suggestions that this procedure produced extra conservative designs, but this guide is intended to always produce a safe, serviceable foundation. Engineers who care to are free to exercise their judgement and to adjust the results in either direction.

SOILS INVESTIGATIONS

It is still mandatory that soils investigation be made on any site to set out the necessary conditions for design. The original recommendation of a minimum of one boring for each isolated site is still valid, but many insuring agencies have specified at least two borings in areas where expansive clay is found. Large sites and subdivisions will need a specific planned program utilizing several borings. Subdivisions will usually average about one boring for every 3 or 4 contiguous lots. Borings should be a minimum of 15 feet deep in most cases, and in some instances will need to be deeper. The soils Engineer should be sure to obtain adequate information to cover any grading changes which can be anticipated. Fill should be identified and noted. Uncompacted fill placed on a site, and improper drainage have been found to be the largest contributors to unsatisfactory foundation performance. Either one or both are guarantees of foundation problems.

During the last 22 years, many alternatives to an adequate on-site investigation have been proposed; soils maps, adjacent data, guesses, and something called a "max design". A "max design" is supposedly a design for the maximum soil condition in the area. How is that known unless an on-site investigation has been done? That is another name for a guess.

What remains true is that the performance of the slab is influenced primarily by the underlying soil. If the severity of the soil is underestimated, the foundation will not be satisfactory. It is therefore essential to know what type soil conditions exist, and that can only be known through an adequate site investigation.

LOADING CONDITIONS

For one, two, and even three story wood frame construction such as homes and small commercial buildings, the assumption of uniform load works well with the design equations. If there are large concentrated loads or numerous columns, attention must be paid to the location of stiffening beams or thickened areas of the slab so that the load can be spread out. Buildings which are carried totally on columns need a different analysis from a uniform loading assumption.

DESIGN ASSUMPTIONS

The design procedure presented originally by The Building Research Advisory Board (B.R.A.B.) in their Report 33, assumed a loss of support at the edges (Fig 1a) and a loss of support at the center (Fig 1b).

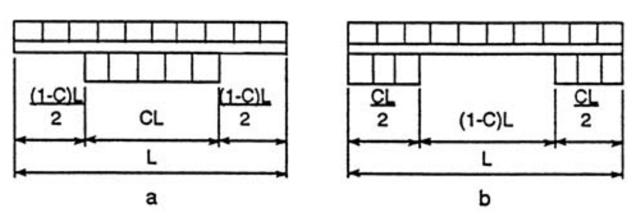

Figure 1

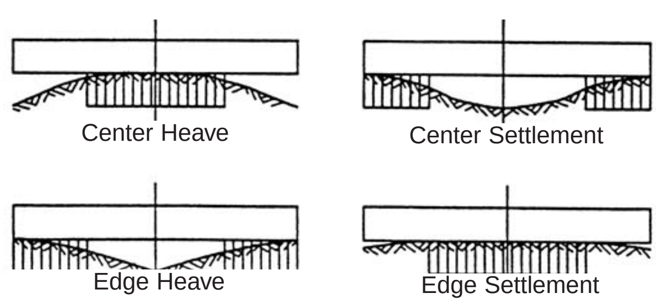

Figure 2

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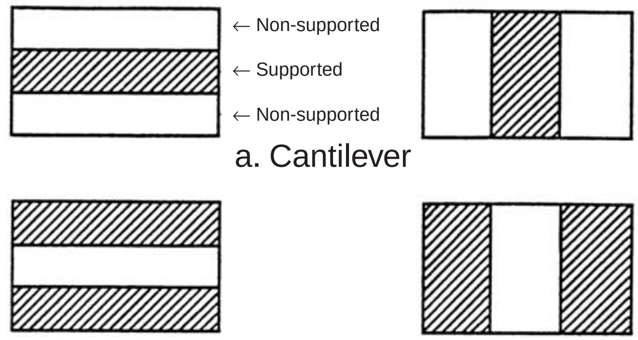
These conditions approximated the conditions of center heave or edge settlement and center settlement or edge heave as shown in Figure 2.

By making some simplifying assumptions it was possible to analyze the foundation slab by applying the loading conditions in both the long and short directions (Figure 3).



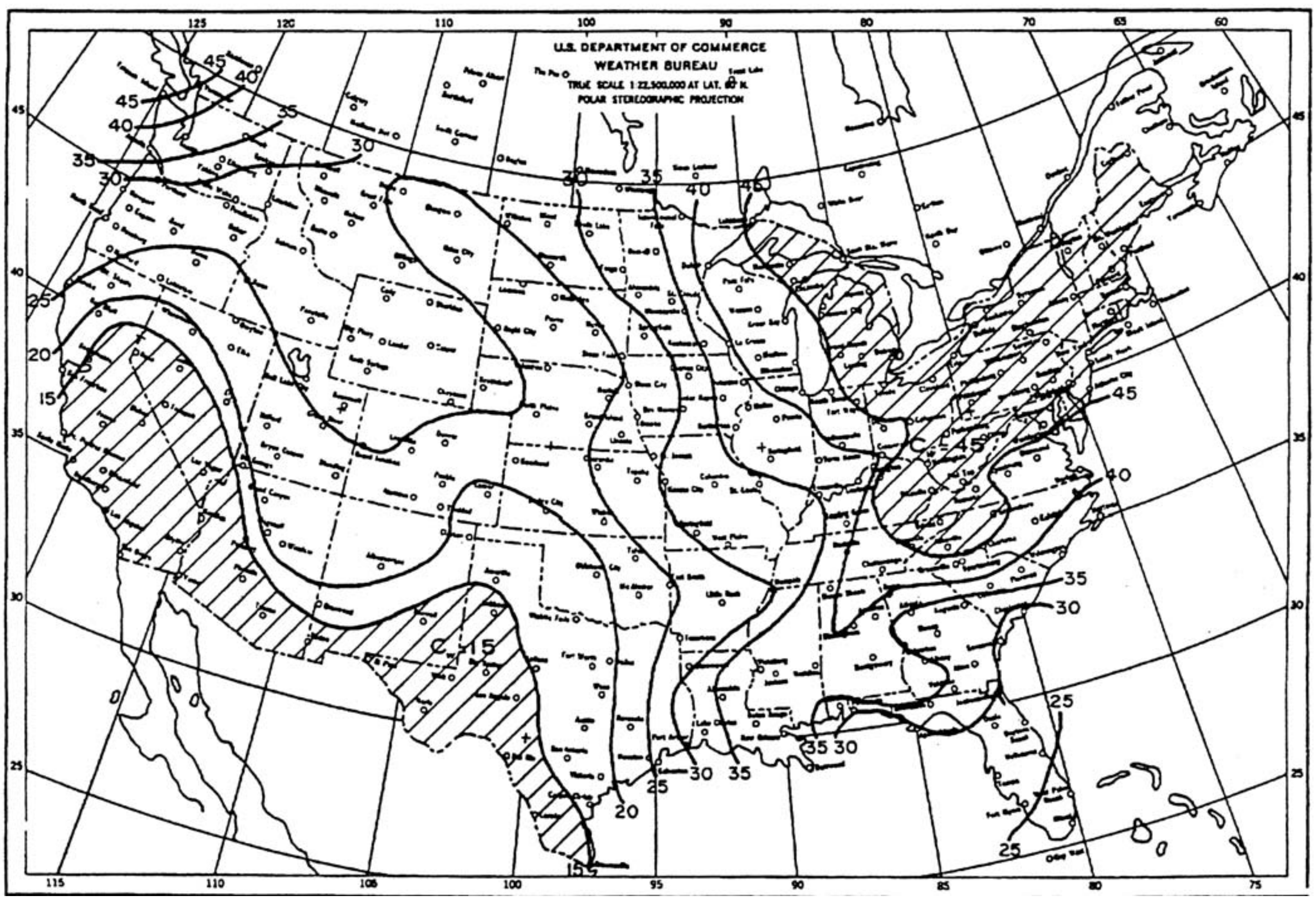
a. Cantilever

b. Simple Span Beam

Figure 3

GEOGRAPHIC CONSIDERATIONS

BRAB utilized the Climatic rating (see Figure 4) of the locality to reflect the stability of the moisture content in an expansive soil. While there are other methods of accounting for the seasonal moisture change potential, this system has seemed to work well.



Climatic Ratings (C_W) for Continental United States

Figure 4

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DESIGN LENGTH

Looking at the various loading conditions above and slabs in the field, it became apparent that the foundations were very sensitive to the changes at the edges. It was decided that a cantilever distance, (l_c) would be used as a basis for this design procedure to replace the L(1-C) utilized by BRAB. Figure 5 gives a cantilever design length for a given soil condition (PI) in a given climatic rating (C_W).

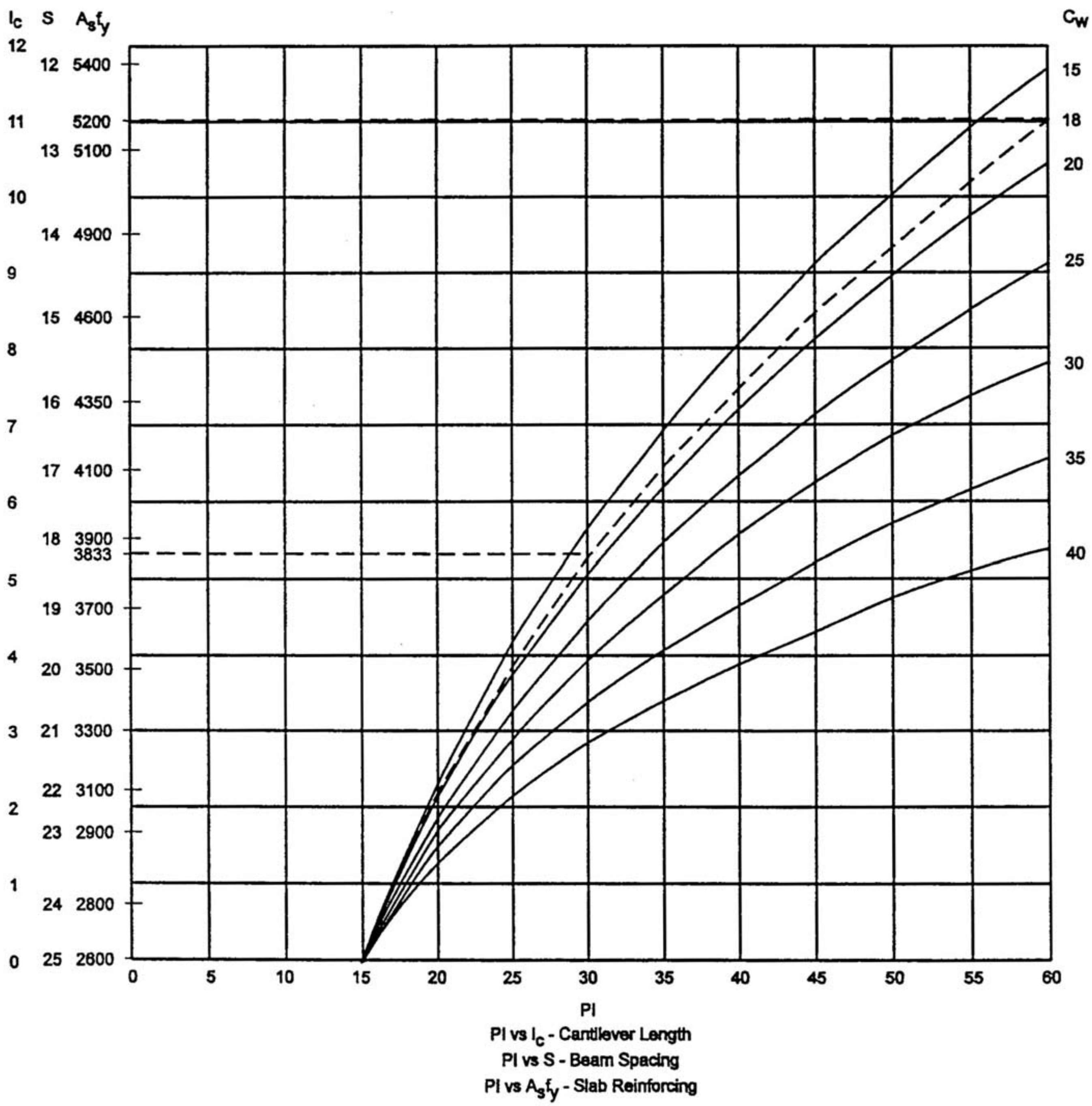

Figure 5

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It seems apparent that the size of the foundation must also be considered. The values given in Figure 5 for the cantilever length are for large slabs. Figure 6 gives a modification coefficient which will adjust the cantilever length for smaller slabs depending on the slab size.

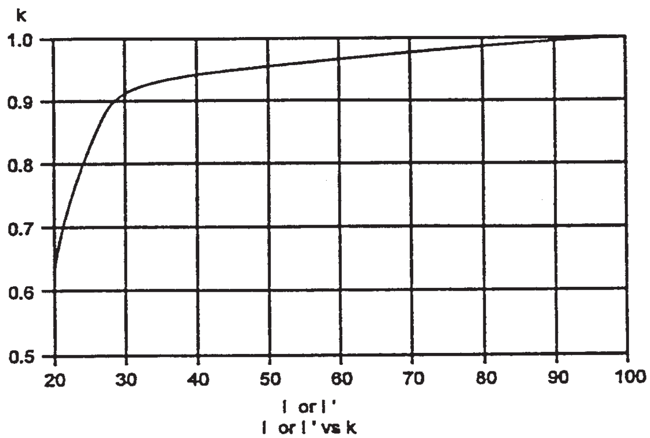


Figure 6

SOIL CONDITIONS

The design procedure shown in this report is based on the use of the "effective P.I." (PI_0). It has long been known that the Plasticity Index (PI) of the soil can be used as an indicator of the Potential Volumetric Activity of a given soil. It has the added advantage of being a test which is familiar and inexpensive to perform.

Obviously, different soils have different PIs, and the PI may change with depth at any one location. To account for this, the design procedure first calculates an "equivalent" or "weighted" PI. It is necessary to use the weighing system shown in Figure 7 to be compatible with this design procedure.

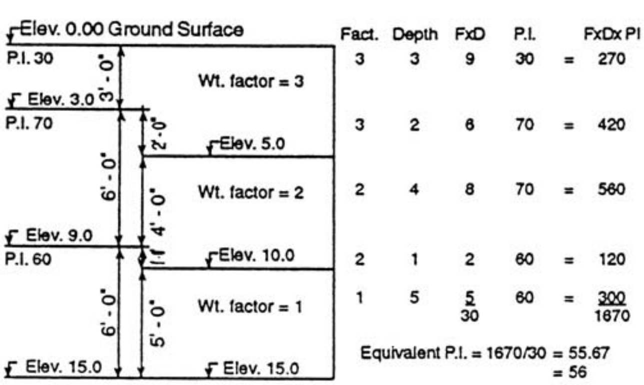


Figure 7

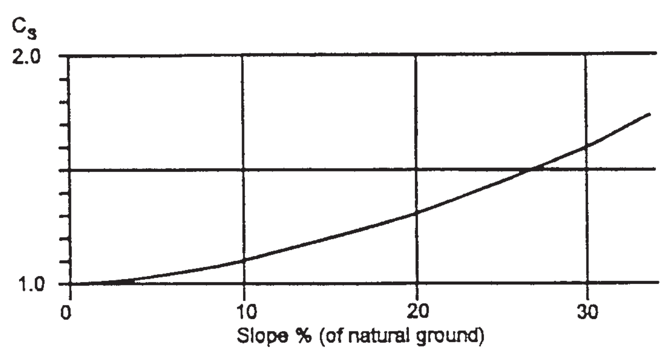
This weighing method gives more attention to the upper soils where the soil would have the opportunity for more activity, and reduces the activity potential with depth due to confining pressure and protection from seasonal moisture changes, etc. This is not the only way to weight this effect, but it has proved to be very satisfactory, and must be used for this procedure.

There are instances where this weighing system might give unconservative results. One would be where the underlying formations might contain sand stringers or are overlaid by porous sand which would provide quick, easy routes for water to reach any underlying or interbedded expansive clays.

A second case would be where highly expansive clays overlaid a rock formation. Using a zero (0) PI. for these rock layers can reduce the equivalent P.I. excessively, making it appear to be a very stable site. It is recommended that to eliminate this problem, a minimum P.I. of 15 be used for any layers which have little or no P.I.

OTHER PARAMETERS OF CONCERN

Other factors to be considered are slope and degree of consolidation. Figures 8 and 9 present modification coefficients to be used with the "equivalent" PI to obtain the "effective" PI.



Slope of natural ground vs. Slope Correction Coefficient

Figure 8

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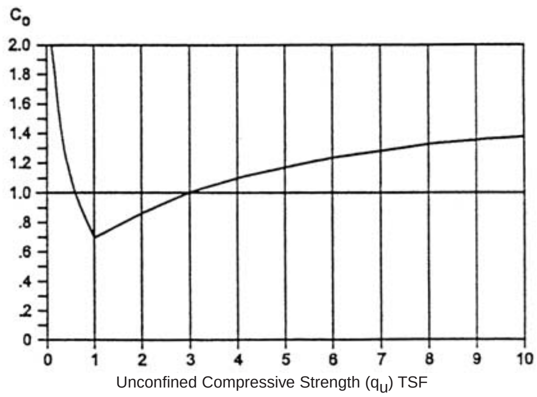


Unconfined Compressive Strength vs. Consolidation Correction Coefficient

Figure 9

The effective PI then is:

PI_o=equivalent PI x C_5 x C_o

Where: C_5 is the slope correction coefficient

C_o is the consolidation correction coefficient

As an example: assume -

Equivalent (or weighted) PI	=	30
10% ground slope C_5 (Fig. 8)	=	1.1
6 TSF Unconfined C_o (Fig. 9)	=	1.2

PI_o = 30 x 1.1 x 1.2 = 39.6

Use an Effective Plasticity Index of 40 for design purposes

HOUSE GEOMETRY AND LOADS

It is best to calculate the total weight of house and foundation, but in lieu of that, or as a starting point it is possible to use the following for most conventional wood frame houses with no unusual features (tile roofs, floors, high masonry loads, etc).

1 story - 200 lb/sq.ft.
2 story - 275 lbs/sq.ft.
3 story - 350 lbs/sq.ft.

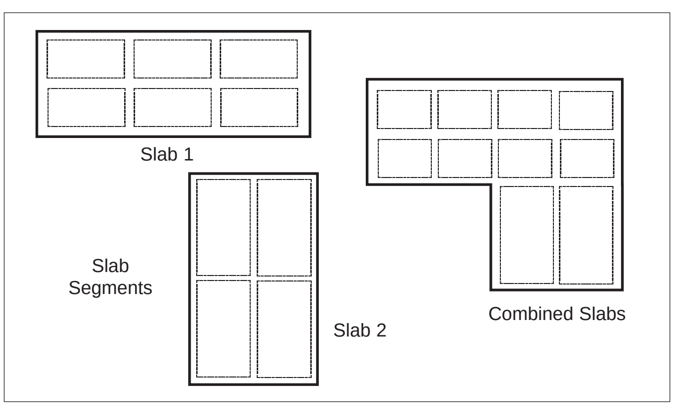


Figure 10

Most houses can be subdivided into several rectangles and each section then be analyzed and then overlaid as shown in Figure 10.

To begin the analysis the number of beams must be determined. Sometimes the geometry of the house will dictate the number of beams (N) required, sometimes the following equation will be used.

$$d = \sqrt[3]{\frac{664\ MI_C}{B}}$$

Where: d = Beam depth, in (mm)
B = Sum of all widths, in (mm)
M = Moment, kip-ft (N-m)
I_C = Cantilever length, ft (m)

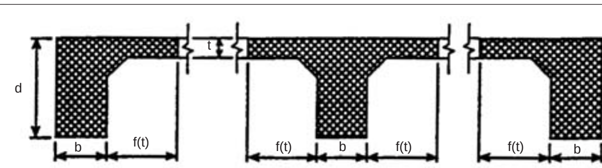
$$N = \frac{L'}{S} + 1$$

Where: S = Spacing ft (m) from Fig. 5
L' = width of slab, ft (m)

Once N is known, a very good first approximation of the depth of the beams can be determined by the equation:

Using these equations yields a starting point with N number of beams, b inches wide and d inches deep which will give a Moment of Inertia (Iin^4) adequate to limit deflection to the order of magnitude of 1/480. This deflection ratio is greater than the usual 1/360, but it usually furnishes beam depths which allow the reinforcing requirement to be two or three bars of moderate size top and bottom. Of course, if the reinforcing requirement is still extremely large, try deepening all or some of the beams to lessen the reinforcing required.

In calculating the actual I of the slab, the sections shown in Figure 11 should be used. As can be seen, the exterior beams can be deepened, or all beams can be deepened. It is felt that deeper exterior beams are more effective, but as long as the slab is kept symmetrical it does not seem to matter.



Effective Width of “T” Beams

Figure 11

DESIGN CALCULATIONS

Now that the conditions have been defined, the following formulas can be used to calculate the moment, deflection and shear.

$$M = \frac{wL' (l_c)^2}{2}$$

$$\Delta = \frac{w (l_c)^4 L'}{4E_c I}$$

$$V = wL' l_c$$

Where: M = Moment + or -, kip-ft (N.m)

Δ = Deflection, in (mm)

V = Total shear, lbs (kg)

w = Unit weight, psf (kg/m^2)

L' = Width of slab, ft (m)

1_c = Cantilever, (l_c k) ft (m)

E_c = Creep Modulus of Elasticity of concrete, psi (MPa)

I = Moment of Inertia, in^4 (mm^4)

Naturally, these calculations will be performed in both the long and short directions.

TEMPERATURE AND SHRINKAGE REINFORCEMENT FOR CRACK CONTROL

The greatest number or reported complaints comes in the form of "cracked slabs". Of course all concrete will crack. Shrinkage crack prevention has spawned a plethora of papers, documents and books. The engineering community understands shrinkage cracking for the most part, but the general public sees each crack as a "structural failure". It is therefore very important to properly address the subject of minimum reinforcing to minimize shrinkage cracking and control crack widths.

The amount of reinforcing needed to control crack formation and width has been found to increase with the expansive potential of the site. Over the years greater need has developed to provide crack control to alleviate homeowners worries. When the beam spacings are near those shown in Figure 5, the minimum reinforcing shown also in Figure 5 is usually adequate. While this will not prevent shrinkage cracking, it will provide adequate reinforcing to hold cracks to a minimum width during deflection. In the field, actual deflection is a function of the expansive nature of the soil, and the stiffness of the slab, so the soil and the beam spacing together influence the deflection. Since the beam spacing is based on the soil (PI) and climate (C_w), the minimum slab reinforcement can also be based on the same factors.

HIGH STRENGTH WELDED WIRE REINFORCEMENT

The use of welded wire reinforcement in concrete has a long history. For this procedure it is strongly recommended that sheets of welded wire, plain or deformed be used. This will provide positive placement in the slab. Welded wire reinforcement sheets can be placed with the same degree of accuracy as tied reinforcing bars. Sheets with larger wires and wider spacing are more readily available, and are easily positioned. The use of high strength welded wire has been accepted by code and some real economies can now be realized, not only in material costs, but in placement costs.

Use of WWR actually provides the engineer a large number of choices as can be seen by the comparison below. Assuming a moderate soil condition and climatic conditions noted, the reinforcing in Chart 1 would be acceptable.

On higher PI soils, it would seem advisable to go to heavier slab reinforcing, even though the stiffness of the slab should be such that cracks would not tend to open any more than at lower PIs. To see how that would look for a higher PI soil, compare Chart 1 to Chart 2.

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COMPARISON OF REINFORCING (1)

PI=60 $C_w = 18$ $A_8 f_y = 3833$

Yield Stress f_y	A_8	Size (W -D)	Spacing**	Style
60000	.064	W6.4	12"O.C.	12x12-W6.4xW6.4
65000	.059	W5.9	12"O.C.	12x12-W5.9xW5.9
70000	.055	W5.5	12"O.C.	12x12-W5.5xW5.5
75000	.051	W5.1	12"O.C.	12x12-W5.1xW5.1
80000	.048	W4.8	12"O.C.	12x12-W4.8xW4.8

Chart 1

COMPARISON OF REINFORCING (2)

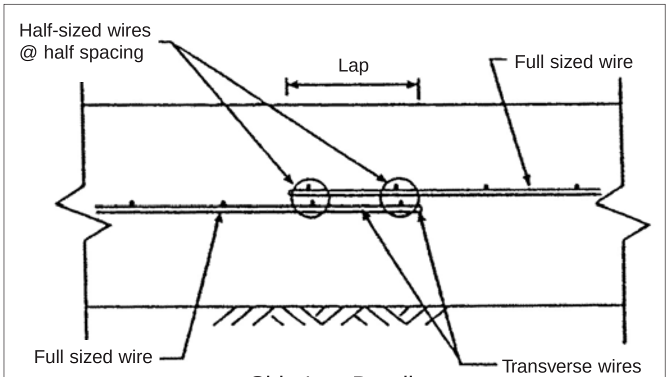
PI=60 $C_w = 18$ $A_8 f_y = 5200$

Yield Stress f_y	A_8	Size*	Spacing** (W-D)	Style
60000	.086	W8.6	12"O.C.	12x12-W8.6xW8.6
65000	.080	W8.0	12"O.C.	12x12-W8.0xW8.0
70000	.074	W7.4	12"O.C.	12x12-W7.4xW7.4
75000	.069	W6.9	12"O.C.	12x12-W69xW6.9
80000	.065	W6.5	12"O.C.	12x12-W6.5xW6.5

Chart 2

These values will approximate requirements of ACI 318, which allows for designs with yield strength up to 80,000 psi.

Use of the higher yield strengths will result in savings due to steel weight. Further savings can be realized by utilizing small edge wires closely spaced as shown in Figure 12. Savings will vary with specific areas, but some studies have shown that for each 5000 psi increase in $f_{y,}$ about 8% in steel weight is reduced. The use of small edge wires closely spaced can save an additional 3% or more. Perhaps the greatest saving will be in placing where costs have been reported to be reduced 50% and more over other conventional steel reinforcing.



Side Lap Detail

Figure 12

A DESIGN EXAMPLE

This design example utilizes welded wire reinforcement for slab-on-ground foundations over soils with high PI values:

Given: PI = 60
C_w=18
$A_8 f_y$ = 5200 lbs (f_y = 75,000psi)
Slab Thickness = 4"

Then: $A_8 = \frac{0.0018 \times 60{,}000}{75{,}000} \times (4 \times 12) = 0.069$ in.²/ft of concrete cross section

Check strength level required: $A_8 f_y$ = 75.000 x 0.069 = 5175 = 5200 OK

CONCLUSIONS

This design procedure, which has been in use about 37 years at this time, has produced satisfactory foundations for single family housing and small commercial applications. This update is meant to make it easier for the consultant to use by combining several tables into one (Fig 5). The Effective PI, and the Climatic Rating are all that need be known to obtain a cantilever length for design.

This paper is a condensation of more detailed work. Engineers may obtain copies of the original work by contacting the WRI. Copyright, Wire Reinforcement Institute Wire Reinforcement Institute 942 Main Street, Suite 300, Hartford, CT 06103 Phone: 800 522-4WRI(4974) • Fax: 860 808-3009 THE AUTHOR Walter L. Snowden, P.E. Austin, TX, 512-338-0431 or 512-338-1804

* W = plain wire, also can be prefix D for deformed wire.

** Wire spacings are available in 2" to 18" in either or both longitudinal and traverse directions. Contact individual welded wire producers for specific styles and spacings of WWR