WALLACE

DEPARTMENT OF THE ARMY
SOUTHWESTERN DIVISION, CORPS OF ENGINEERS
1114 COMMERCE STREET
DALLAS, TEXAS 75242-0216

REPLY TO
ATTENTION OF

CESWD-ED-TS/G (415a)

29 JAN 1989

MEMORANDUM FOR:

Commander, Albuquerque District, ATTN: CESWA-ED
Commander, Fort Worth District, ATTN: CESWF-ED-DT
Commander, Galveston District, ATTN: CESWG-ED
Commander, Little Rock District, ATTN: CESWL-ED
Commander, Tulsa District, ATTN: CESWT-ED

SUBJECT: Design Criteria for Ribbed Mat Foundations

1. This letter supersedes criteria letter, SWDED-TS/G, 23 Dec 1986, SAB.

2. The enclosed criteria shall be used for design of all ribbed mat foundations. This criteria has been revised to conform with the definition of swell pressure (soil-beam interface pressure) as presented in criteria letter, SWDED-G, 16 Apr 1987, subject: Criteria for Developing Geotechnical Design Parameters for SWD Ribbed Mat Design Methodology. Also, clarification has been provided for application of the PTI design method to family housing.

3. This criteria is furnished to addressees only.

FOR THE COMMANDER:

Encl

ARTHUR D. DENYS, P.E.
Chief, Engineering Division

DESIGN OF RIBBED MAT FOUNDATIONS

BY

JOSEPH P. HARTMAN

AND

B. H. JAMES

U.S. ARMY CORPS OF ENGINEERS

SOUTHWESTERN DIVISION

DALLAS, TEXAS

REVISED

JANUARY 1988

CONTENTS

PART I - GENERAL REQUIREMENTS FOR RIBBED MATS

1. REFERENCES.

1.1 Engineering Instruction Manual, Corps of Engineers, Southwestern Division, (latest edition).

1.2 "Criteria for Selection and Design of Residential Slabs-on-Ground," Building Research Advisory Board (BRAB), Report No. 33 to the Federal Housing Administration, 1968.

1.3 "Design and Construction of Post-Tensioned Slabs-on-Ground," Post Tensioning Institute (PTI), 1980.

1.4 TM 5-818-7, Foundations in Expansive Soils, Corps of Engineers, 1983.

1.5 Letter, SWDED-G, 16 April 1987, Criteria for Developing Geotechnical Design Parameters for Ribbed Mat Design Methodology (Criteria Letter XV 7-12).

2. BACKGROUND. Ribbed mat foundations consist of a thin slab on grade which acts monolithically with a grid of stiffening beams beneath the slab. The beams (ribs) are cast in trenches dug in the foundation soil. Ribbed mats combine the economic advantages of shallow foundations with the performance advantages of monolithic floors. Ribbed mats are especially useful for minimizing differential foundation movements in areas with expansive soils.

3. DESIGN METHODS.

3.1 EXPANSIVE SOILS.

3.1.1 Behavior.

3.1.1.1 Center Lift. In the center lift condition the soil near the edge of the slab drops in relation to the soil near the center. This is due to moisture retention by the interior soils and the drying and shrinking of perimeter soils. As this occurs, the perimeter soil provides less support for the edge of the slab which then acts as a cantilever. This is illustrated in Figure A1 of Appendix A.

3.1.1.2 Edge Lift. In the edge lift condition the soil near the edge of the slab rises in relation to the soil near the center. This is due to the increasing moisture content and subsequent swelling of soil near the edge. The swelling soil raises the edge of the slab, causing some of the slab to lift off the soil. Interior loads cause the slab to sag and recontact the soil at some interior location. The slab thus tends to act as a beam, simply supported by the soil at the edge, and by soil towards the center of the slab. The amount of support at the center depends on numerous parameters such as interior loads, rib bending stiffness, soil swell pressures, and the magnitude of soil swelling. Typical edge lift behavior is illustrated in Figure A3 of Appendix A.

3.1.2 **SWD Method.** All ribbed mats on expansive soils, except for family housing, shall be designed in accordance with the provisions of Part II of this report. Ribbed mats for family housing may be designed in accordance with Part II or paragraphs 3.1.3 or 3.1.4.

3.1.3 **PTI Method.** The PTI method (reference 1.3) may only be used for design of family housing foundations on expansive soils. Specifically, slab width (short dimension) should not exceed 40 feet, rib depths should not exceed 30 inches, loading should consist only of perimeter loads and light interior distributed loads ($DL+LL \leq 100$ psf), soils should be fairly weak in-situ materials with no extensive substitution of non-expansive fill. When using the PTI method, the following provisions shall apply: Rib spacing shall not exceed 15 feet; concrete tensile stress shall not exceed $4\sqrt{f'c}$; the minimum effective prestress shall be 100 psi.

3.1.4 **BRAB Method.** The BRAB report (reference 1.2) may only be used for design of foundations for family housing. However, the PTI method is preferred, since the BRAB method may produce unreasonable results for large foundations.

3.1.5 **Computer Method.** In lieu of paragraph 3.1.2, ribbed mats may be designed using appropriate computer programs. Such programs must be capable of modeling the variable soil swell due to moisture changes, and the non-linear soil-structure interaction near the perimeter of the foundation. One such computer program is CBEAMC, program X0050 in the Corps of Engineers Civil Engineering Library.

3.1.6 Load Factors. When using the above methods to design ribbed mats for center lift and edge lift conditions, load factors may be multiplied by .75 (strength method) or allowable stresses may be increased by one-third (working stress method). This provision does not apply to the allowables given for the PTI method, since those allowables have already been increased from the usual provisions of ACI.

3.2 **NON-EXPANSIVE SOILS.** Ribbed mat slabs on non-expansive soils need not be designed for bending due to center lift or edge lift conditions. Beam on elastic foundation analyses may be used to determine the effects of concentrated loads on ribs, or ribs may be designed as conventional strip or spot footings.

3.3 SOIL PROPERTIES. Soil properties for design of ribbed mats will be provided in the Foundation Design Analysis by the Corps of Engineers. Criteria for developing these properties is included in reference 1.5. The properties necessary for design in accordance with paragraph 3.1.2 consist of the following, which are defined in Appendix A:

- qa - allowable bearing pressure
- k - subgrade modulus
- Ym - soil heave
- Lm - edge moisture variation distance
- Psw - pressure of swelling soil acting on perimeter rib

4. MINIMUM REQUIREMENTS.

4.1 SUBGRADE PREPARATION. A vapor barrier, capillary water barrier, and a minimum of 18 inches of non-expansive fill will normally be used beneath ribbed mats. Additional non-expansive fill will often be used to lessen the effects of highly expansive soils. These requirements will be detailed in the Foundation Design Analysis.

4.2 SLAB. For family housing and other small lightly loaded buildings a 4 inch slab may be used. For other buildings the minimum slab thickness will be 5 inches. Minimum slab reinforcing shall be 0.2 percent. Where slabs are subjected to vehicular loading they must be designed for the maximum wheel load, similar to paving. Use 650 psi flexural strength concrete for slabs subject to wheel loads.

4.3 GRID GEOMETRY. Ribs should be located to form a continuous grid. Rib spacing should not exceed 20 feet in expansive soils, or 25 feet in non-expansive soils. Locations of ribs should conform to significant wall and column loads, and may be used to resist thrusts from rigid frame reactions. Ribs should be provided around large openings in the slab. In expansive soils diagonal ribs are required at exterior corners.

Expansion joints should be provided at 250 foot intervals, and should also be used to break irregularly shaped buildings into rectangular segments. Foundations for family housing do not require expansion joints due to irregular shapes.

4.4 RIB SIZE. Minimum rib depth is 20 inches. Rib depths should usually not exceed 3 feet to minimize construction difficulties related to placing reinforcement and maintaining trench walls. If deeper ribs are used, rib width should also be increased. Minimum rib width is 12 inches except for family housing foundations, where 10 inch ribs may be used. Sufficient rib width must also be provided to transfer wall and column loads to the soil as strip footings. The allowable soil bearing capacity may not be exceeded when considering the width of the rib plus an effective slab width on each side of the rib. The effective slab width for bearing is limited to the thickness of the slab. At column locations an alternate is to provide fillets at rib intersections, sufficient to act as spot footings for column loads.

4.5 RIB CAPACITY. Concrete should have a minimum compressive strength of f'c=3000 psi at 28 days. Reinforcing shall be grade 60, except ties may be grade 40. Minimum reinforcing ratio (As/Ag) shall be .0033 top and .0033 bottom, this may be reduced to .005 total in non-expansive soils. Use #3 ties at 24 inches, minimum. These minimums should be sufficient for shrinkage stresses and for unpredictable soil behavior.

4.6 PRESTRESSED MATS. For prestressed ribbed mats, not designed per PTI, all the above minimum requirements apply except that slab and rib top reinforcement may be deleted and replaced by appropriate post-tensioning strands. Mild steel shall still be provided in the bottom of ribs. Minimum effective prestress shall be 100 psi on the gross area of the slab, including effects of subgrade friction as calculated by the PTI method, reference 1.3. Concrete tensile stress shall be limited to $3\sqrt{f'c}$ and shear stress limited to $1.1\sqrt{f'c}$. A one-third overstress may be allowed per paragraph 3.1.6.

4.7 CONSTRUCTION DETAILS.

4.7.1 Conventionally Reinforced. Construction joint spacing should not exceed 50 feet in either direction. A horizontal construction joint may be provided in the ribs at the base of the capillary water barrier when unstable trench walls may cause construction difficulties. However, this joint is discouraged because of increased potential for shrinkage cracks in the slab.

4.7.2 Prestressed. Construction joint spacing shall not exceed 75 feet in either direction. Tendons within each placement shall be stressed to 15 percent of the final prestress not more than 24 hours after the concrete has attained sufficient strength to withstand the partial prestress. Other construction procedures for prestressed ribbed mats shall conform to reference 1.3.

4.7.3 Contractor Designs. Ribbed mat foundations may be designed as prestressed or conventionally reinforced as selected by the engineer. The plans and specifications shall not include the option of changing the ribbed mat from one type to another. The reason for this prohibition is that design parameters (e.g., moments of inertia) may be dependent on the type of ribbed mat being designed and may affect calculated shears and moments. This does not prohibit revisions of the slab type as a result of contractor value engineering proposals. However, such revisions must include a complete design of the ribbed mat foundation using appropriate design parameters in accordance with this report.

PART II - ANALYSIS OF RIBBED MAT FOUNDATIONS ON EXPANSIVE SOILS

1. SCOPE. This part of the report contains the basic rules for design of ribbed mats in expansive soils. This method may be used to predict shears, moments and deflections in ribs subject to soil movement due to changing moisture content. For a commentary on the design method refer to Appendix A; for example design calculations refer to Appendix B. The design method from Part II should be used in conjunction with the "minimum requirements" for ribbed mats, as presented in Part I.

2. GENERAL

2.1 NOTATION.

C	=	Correction factor for equivalent cantilever length
D	=	Beam deflection (IN)
I	=	Moment of inertia per foot, I=Ir/S (IN^4/FT)
Ir	=	Moment of inertia of rib (IN^4)
* k	=	Modulus of subgrade reaction (PCI)
Lo	=	Basic length of cantilever (FT)
Lc	=	Equivalent length of cantilever, center lift (FT)
Le	=	Equivalent length of simple beam, edge lift (FT)
Li	=	Distance from perimeter to location of interior load (FT)
* Lm	=	Edge moisture variation distance (FT)
Lb	=	Width of soil bearing at perimeter, edge lift (FT)
M	=	Bending moment per foot (FT-LB/FT)
Mr	=	Bending moment per rib, Mr=MxS (FT-LB)
Pi	=	Interior load (PLF)
Pp	=	Perimeter load (PLF)
* Psw	=	Pressure of swelling soil on perimeter rib (PSF)
R	=	End reaction at perimeter for equivalent simple beam (PLF)
S	=	Rib spacing (FT)
w	=	Uniform load (PSF)
V	=	Shear per foot (LB/FT)
Vr	=	Shear per rib, Vr=VxS (LB)
* Ym	=	Soil heave (IN)
θ	=	Rotation of support of equivalent cantilever (RAD)

* q_a = ALLOWABLE BEARING PRESSURE (PSF)

2.2 UNITS. The equations presented in section 3 are written for units as defined in the above notation. If other units are used the equations must be modified appropriately.

2.3 RIB DEFINITIONS. Ribs are defined as perimeter, transverse or diagonal as shown in Figure 1. Note that transverse refers to ribs parallel to either axis of the building.

* VALUES NEEDED FROM GEOT[illegible]

FIGURE 1 - RIB DEFINITIONS

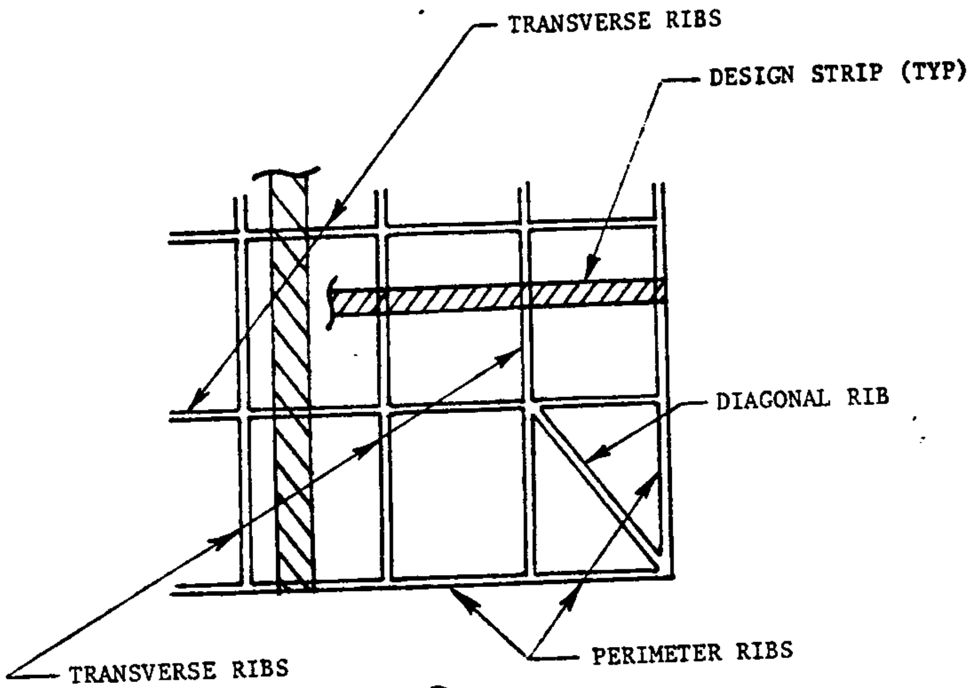
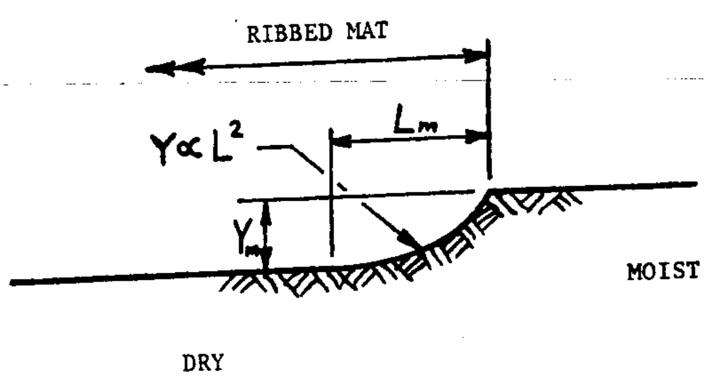


FIGURE 2 - SOIL EDGE PROFILE



2.4 STRIP ANALYSIS. The analysis is based on a strip assumption, ignoring the effects of the grid configuration of the ribs. The formulas and examples presented below are for an equivalent 1-foot strip, using "per foot" values for loads and stiffness.

2.5 SOIL EDGE PROFILE. For edge lift the maximum swell occurs at the perimeter and decreases rapidly toward the interior. The soil profile is assumed to be parabolic (in the unloaded condition) and is illustrated in Figure 2.

3. ANALYSIS METHOD.

3.1 TRANSVERSE RIB - CENTER LIFT.

3.1.1 General. Center lift analysis is based on an equivalent cantilever beam to determine moments, shears and deflections.

3.1.2 Moment. The length of the equivalent cantilever can be calculated as:

$$Lc = C \times Lo$$

where: $Lo = 2.3 + .4\ Lm$

$$C = .8\ Ym^{.12}\ I^{.16} / Pp^{.12}$$

The maximum moment may then be calculated from statics using conventional cantilever formulas such as:

$$M = Pp\ Lc + 1/2\ w\ Lc^2$$

The moment can then be assumed to be constant for a distance Lc/2 and then to decrease linearly to zero at a distance 5Lc from the perimeter. To obtain the design moment for a given rib, multiply the calculated per-foot moment by the appropriate rib spacing (Mr = M x S).

3.1.3 Shear. The maximum shear may be calculated from statics using the same equivalent cantilever as for moment.

$$V = Pp + w\ Lc$$

The shear may then be assumed to decrease linearly from V at the cantilever support, to zero at a distance 5Lc from the perimeter. To obtain the design shear for a given rib, multiply the calculated per-foot shears by the appropriate rib spacing (Vr = V x S).

3.1.4 Deflection. Deflection at the perimeter is the sum of three components: bending deflection of the equivalent cantilever, vertical translation of the cantilever support, and rotation of the cantilever support. Rotation of the support may be calculated as:

$$\theta = M^{1.4} / 9800\ I\ k^{.5}$$

The perimeter deflection is then:

$$D = .11 + 12\ Lc\ \theta$$

where .11 inches is an approximation for the support translation plus the cantilever bending, and (12 Lc) is the length in inches.

Use the deflection calculated above to compare with allowable deflection. The allowable deflection may be determined by using 4Lc as the length between points of zero and maximum deflection.

3.2 TRANSVERSE RIB - EDGE LIFT.

3.2.1 General. Edge lift analysis is based on an equivalent simple beam, supported at the perimeter and at some interior location.

3.2.2 Deflection. The first step in calculating deflection is to determine the length of the equivalent simple beam. The appropriate length depends on many parameters, including the deflection. Therefore, deflection must first be estimated to determine equivalent length, then a deflection is calculated based on that length. The process is repeated until calculated deflection matches the assumed deflection. The equivalent simple beam length may be calculated as:

$$Le = 7.5\ I^{.17}\ Li^{.37}\ D^{.12} / w^{.07}\ Pi^{.11}$$

The perimeter end reaction for this beam may be calculated from statics. For a given case the reaction may be:

$$R = Pp + 1/2\ w\ Le + Pi(Le-Li)/Le$$

The width of soil bearing at the perimeter can be approximated as:

$$Lb = 1.1\ (R/Psw)$$

where Psw is selected from a curve of heave versus bearing pressure, corresponding to the estimated deflection used during this iteration (see reference 1.5).

The edge deflection is found by determining the soil swell at a distance Lb from the perimeter, based on the parabolic swell profile:

$$D = Ym(Lm-Lb)^2/Lm^2$$

When satisfying deflection criteria, use the calculated deflection and equivalent simple beam length.

3.2.3 Moment. Once the simple beam equivalent length has been determined, the bending moments may be calculated based on statics. To obtain rib design moments, multiply per-foot moments by the rib spacing.

3.2.4 Shear. Once the simple beam equivalent length has been determined, the shears may be calculated based on statics. To obtain rib design shears, multiply per-foot shears by the rib spacing. Near the interior support the design shear need not exceed:

$$V = Pi + w(Le-Li)$$

This is due to the effects of the actual distributed soil support, rather than the point support assumed in the simple beam analysis.

3.2.5 Special Cases. If Pi=0 or if Li>Le make the following substitution in the equation for Le:

$$1.4 = Li^{.37} / Pi^{.11}$$

The equation for the simple beam length then becomes:

$$Le = 10.5\ I^{.17}\ D^{.12} / w^{.07}$$

3.3 PERIMETER RIB.

3.3.1 Center Lift. For center lift the perimeter rib will have no support from the soil and must be designed to span between transverse ribs for the perimeter wall loads.

3.3.2 Edge Lift. For edge lift the soil pressure on the perimeter rib will exceed the applied perimeter loads. The perimeter rib must designed to span between transverse ribs for this net upward force.

3.4 DIAGONAL RIB. Diagonal ribs are used to support exterior corners for center lift conditions, if loss of support occurs under both perimeter ribs. Diagonal ribs must be designed to provide the same moment and shear capacity as the larger of the two adjacent transverse ribs.

3.5 INTERIOR RIB. Interior ribs and rib intersections should be located at significant wall and column loads. The ribs can be designed for these loads as strip or spot footings, using beam-on-elastic-foundation methods. Differential soil movement due to moisture change is assumed not to occur except at the perimeter. However, to account for unpredictable interior soil movements, interior ribs must have the minimum size and capacity as required in Part I.

APPENDIX A - COMMENTARY ON PART II

1. SCOPE. Actual behavior of ribbed mats in expansive soils involves complex, non-linear, soil-structure interaction. The best solution for such behavior is provided by computer programs. The hand design method has been developed to approximate such computer results. Hand solutions have been checked by computer analyses; results have been within acceptable limits of error. However, such checks have been made only for a limited range for each design parameter, as shown in Table A1, corresponding to the usual values for military construction within Southwestern Division. If a wider range of parameters is applied to the hand design formulas, the results may be less accurate.

TABLE A1

Parameter	Units	Minimum	Maximum
k	pci	50	200
Ym	in	0.5	3.0
Lm	ft	2	8
I	in^4/ft	750	6000
Pp	lb/ft	1000	5000
Pi	lb/ft	0	5000
Li	ft	6	20
w	psf	100	250
Psw	psf	2000	8000

2. GENERAL.

2.1 NOTATION.

Ir = moment of inertia of rib. For non-prestressed rib mats Ir should be the effective moment of inertia, calculated per ACI 318, Section 9.5.2.3.

k = Modulus of subgrade reaction. This parameter is the ratio of the soil pressure at the base of the concrete and the corresponding settlement. Since modulus values are typically determined by plate-load test at the ground surface, they should be corrected for depth and for footing size (expected high pressure area between concrete and soil). Analyses have indicated that the high bearing pressure area for center lift conditions will occur in an area several feet long parallel to the transverse rib and several feet on each side of the rib. A crude approximation for this area would be 5 feet square. This approximation should be adequate for design, since calcualations are not sensitive to small changes in the modulus of subgrade reaction.

qa = Allowable bearing pressure. This is the safe bearing capacity of the soil at the base of the ribs. A factor of safety of 3.0 is recommended for computing this value.

Lm = Edge moisture variation distance. This represents the distance, inward from the edge of the slab, over which the moisture content of the soil changes. Much judgement is required in determining this value.

Psw = Pressure of swelling soil on perimeter rib. This is the interface pressure between the soil and the base of the exterior rib, due to an increase in soil moisture content. The pressure which can be exerted by the swelling soil is dependent on the amount the surface of the soil is allowed to rise. Therefore Psw is usually presented as a curve of pressure versus heave, as described in reference 1.5 of Part I. The actual upward deflection of the edge of the slab is a complex interaction between swell potential, structural loads, and mat stiffness, all of which combine to determine the interface pressure near the perimeter.

Ym = Soil heave. This is the differential vertical movement of the soil representing either soil heave (edge lift) or soil shrinkage (center lift). The magnitude of Ym is the computed vertical movement of a particle of soil at the ground surface due to a change in moisture content. This value should be based on the accumulation of potential volume changes for the full thickness of the active zone (Za), with no significant loads applied to the foundation. The value of Ym may differ for edge lift and center lift conditions.

Pi, Pp, $\bar{w}$ = Applied loads. Loads should consist of full dead plus live loads, including dead load of the slab and ribs.

2.2 UNITS.

2.3 RIB DEFINITIONS.

2.4 STRIP ANALYSIS. The hand solution formulas have been developed for analysis of an equivalent 1 foot strip. This is convenient for uniform loads and for soil properties, but requires some calculations for appropriate concentrated loads and bending stiffness. Rib stiffness must be divided by rib spacing to get the per-foot stiffness. If column loads exist they must also be divided by the rib or column spacing to provide an equivalent load per foot. If interior wall loads are parallel to the transverse rib, they must be divided by the rib spacing. These calculations are illustrated in Appendix B.

2.5 SOIL EDGE PROFILE. The edge lift condition occurs when increased moisture content swells exterior soils, and this effect extends under the edge of the slab. The center lift condition occurs when soils under the slab are generally moist and seasonal drying occurs on the exterior, again extending under the slab. This causes the soil to shrink away from the edge of the slab.

The analysis method is based on an assumed parabolic swell profile which occurs uniformly along the perimeter. This is a convenient idealization of actual soil behavior, which is certainly more erratic. However, the parabolic profile has better correlation with measured swells than do other possible edge profile assumptions. Note that the soil profile is not used in the hand design formulas for center lift. However, a parabolic profile was used in the computer analyses for center lift, which formed the basis for the hand design forumlas.

3. ANALYSIS METHOD. Many of the formulas for shears, moments and reactions are idealized, assuming Pp and R are exactly at the perimeter and that w extends to the perimeter. These approximations should usually be acceptable, but the formulas may be modified to account for actual load patterns.

3.1 TRANSVERSE RIB - CENTER LIFT

3.1.1 General. Typical behavior of a transverse rib for center lift conditions is shown in Figure A1. This illustrates the soil bearing pressure and the shear, moment and deflection. Note that the effects of the soil movement extend much farther than the moisture variation distance. The moment and shear distribution close to the edge resemble cantilever behavior.

3.1.2 Moment. The extent of significant moments is illustrated in Figure A1. The length of the equivalent cantilever can be taken as a basic length (Lo) which is dependent on the moisture variation distance, times a correction factor (C) which accounts for secondary effects of several parameters. The value of the correction factor will usually be slightly greater or less than unity. The correction factor was developed to permit accurate approximations of computer results. It was developed from the ratios of actual values to usual values for significant parameters. For example, the "usual" values are: Ym = 1 in, I = 1500 in^4/ft, Pp = 3000 lb/ft. Thus:

$$C = (Ym/1.0)^{.12} (I/1500)^{.16} (3000/Pp)^{.12}$$

$$C = .8\ Ym^{.12}\ I^{.16} / Pp^{.12}$$

A similar approach was used to develop all the formulas in Part II which have an exponential format.

3.1.3 Shear. Maximum shear occurs near the support of the equivalent cantilever. The extent of significant shears is illustrated in Figure A1.

3.1.4 Deflection. Formulas for deflection include an assumed concrete modulus of elasticity Ec = 3,320,000 psi, for both center lift and edge lift.

Vertical movement at the perimeter is much greater than the bending deflection of the equivalent cantilever. To predict the deflection it is necessary to consider translation and rotation at the support of the equivalent beam. The most significant component is due to rotation at the support. These components of deflection are shown in Figure A2. The sum of the cantilever bending and the support translation are approximated by the value 0.11 inch. The percent error due to this approximation is negligible when total deflections are large. The percent error is greater when total deflections are small, but then the deflections are not significant anyway.

Allowable deflections (see Part I, reference 1.1) are expressed as a ratio of the difference in vertical movement at any two points, compared to the distance between those points. For example: $D \leq L/600$, where D is the differential displacement. In such formulas it is appropriate to use the point of maximum deflection and a point of near-zero deflection as the two measuring points. For center lift behavior the maximum deflection occurs at the perimeter, and deflections tend to die out at approximately 4Lc (four times the equivalent cantilever length) from the perimeter. Therefore, the ratio D/4Lc is appropriate for comparison with allowable deflections.

3.2 TRANSVERSE RIB - EDGE LIFT.

3.2.1 General. Typical behavior of a transverse rib for edge lift conditions is shown in Figure A3. This illustrates the soil bearing pressure and the shear, moment and deflection. Soil swell lifts the edge of the ribbed mat, which actually rises off the soil for some distance from the perimeter. For shear and moment, this portion of the rib acts as a simply supported beam spanning between soil support at the perimeter and at an interior location.

3.2.2 Deflection. Vertical movement at the perimeter is driven by the tendency of the soil to swell, and is resisted by the downward loads applied on the soil. As the soil swells at the perimeter the slab is lifted off the interior soil. This concentrates soil reactions near the edge, causing very high pressures. The pressures rise so high that they limit the capacity of the soil to swell. Thus, the soil cannot swell as much as it would if not loaded. Deflections can be predicted by balancing the upward force of the soil (the swell pressure times the bearing width) with the downward force of applied loads. This downward force can be determined from statics once an equivalent simple beam length is determined. The method for determining the deflection is shown in Figure A4.

Allowable deflections are expressed as ratios, as discussed in the commentary on paragraph 3.1.4. From Figure A3 it can be seen that the appropriate values for this ratio are the edge deflection and the equivalent simple beam length (D/Lc).

Edge lift deflections are mainly a function of soil properties and applied loads, bending stiffness of the ribs has only a secondary effect. Therefore, it may not be possible to control deflections by increasing the rib stiffness. It may be necessary to accommodate calculated deflections by using a less brittle superstructure or by detailing the superstructure to make it less sensitive to deflections. Or it may be necessary to modify soil properties to minimize the edge heave.

3.2.3 Moment. The moments can be calculated by statics, using the equivalent simple beam. The maximum moment will occur at the point of zero shear. Note that the maximum moment is quite sensitive to the beam length, therefore the iterative solution for deflection and appropriate swell pressure must converge accurately before calculating moments.

3.2.4 Shear. Shears can also be calculated by statics from the equivalent simple beam. Note that shears will reduce gradually to near-zero around the interior end of the beam because of the distributed soil support.

3.2.5 Special Cases. If no concentrated interior load exists, or if it is very far from the perimeter, the formula for the simple beam length must be adjusted as shown. This adjusted formula was also developed to duplicate results from computer solutions.

3.3 PERIMETER RIB.

3.4 DIAGONAL RIB.

3.5 INTERIOR RIB. Potential soil heaves in the interior are unpredicatable and are generally due to localized moisture conditions, for example, due to a leaking pipe. Such conditions cannot be accounted for by design forumlas. Adequate strength and stiffness for such unpredictable heaves should be supplied by the minimum requirements listed in Part I of the report. For interior wall or column loads the interior ribs should be designed in accordance with Part I, section 3.2.

FIGURE A1 - CENTER LIFT BEHAVIOR

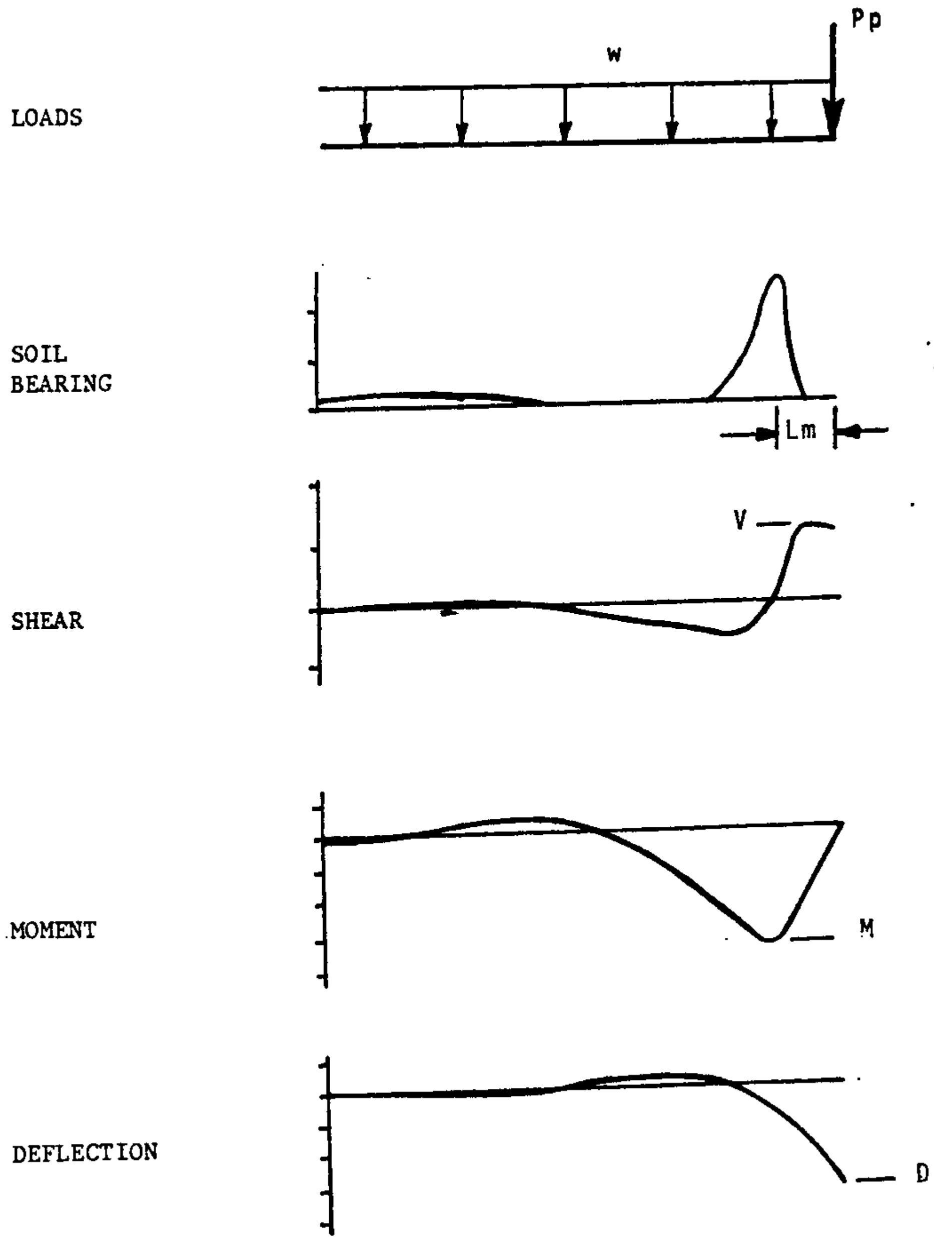
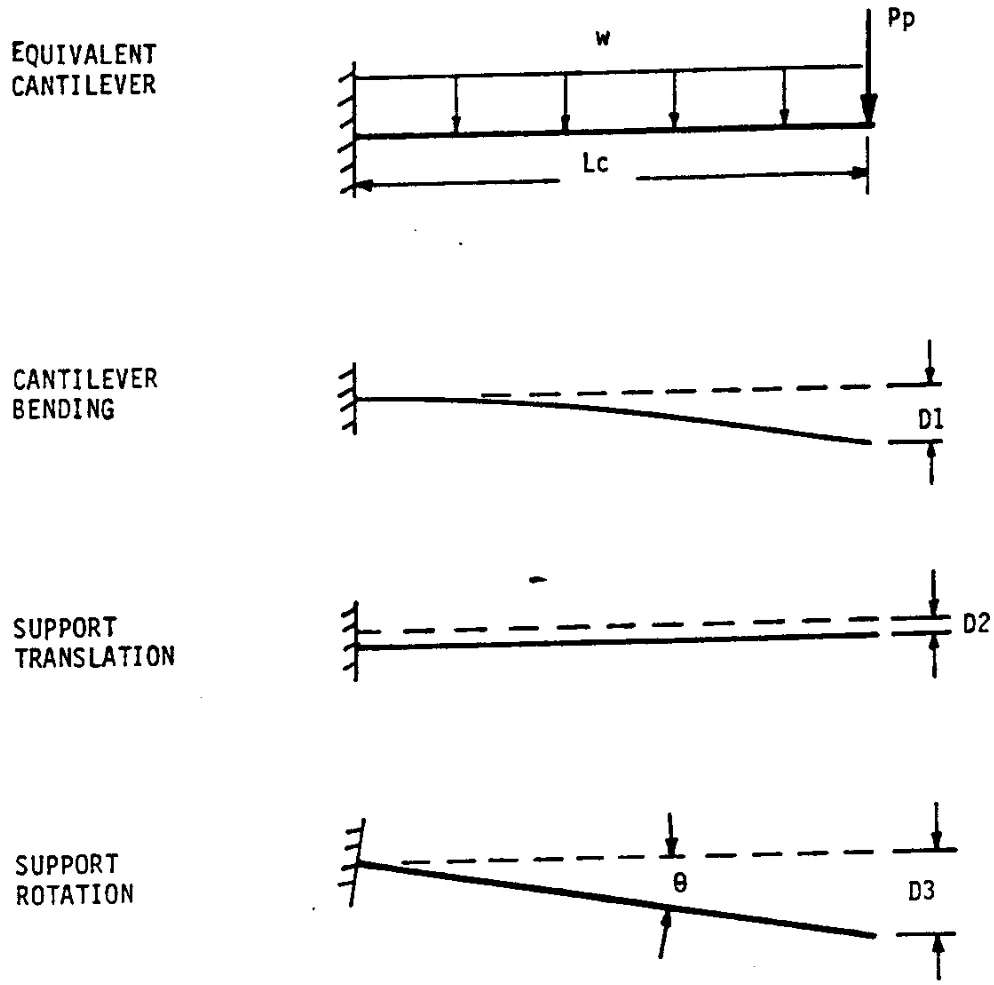


FIGURE A2 - CENTER LIFT DEFLECTION



$$D = D1 + D2 + D3$$

$$D1 + D2 = .11$$

$$D3 = 12\ Lc\ \theta$$

FIGURE A3 - EDGE LIFT BEHAVIOR

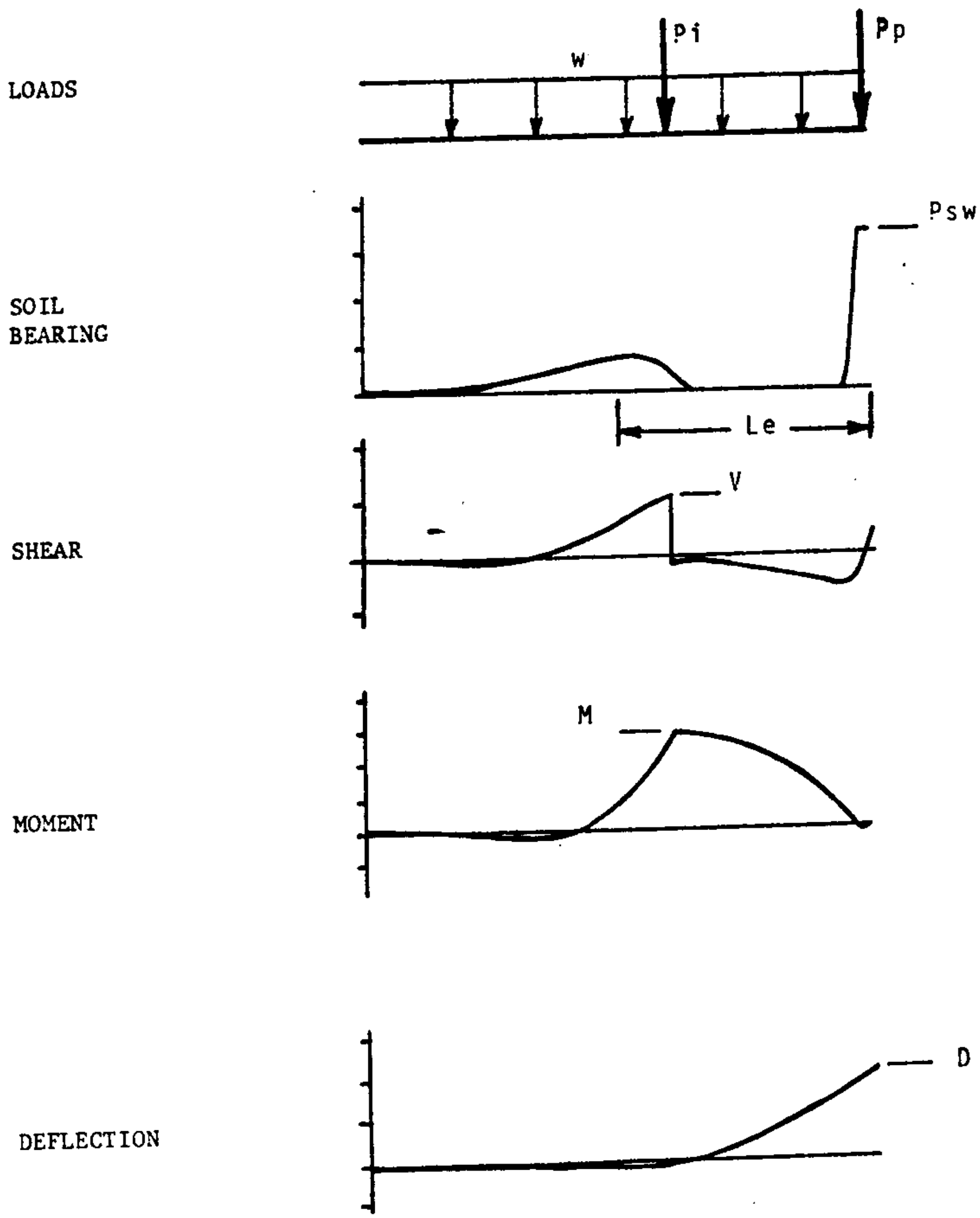
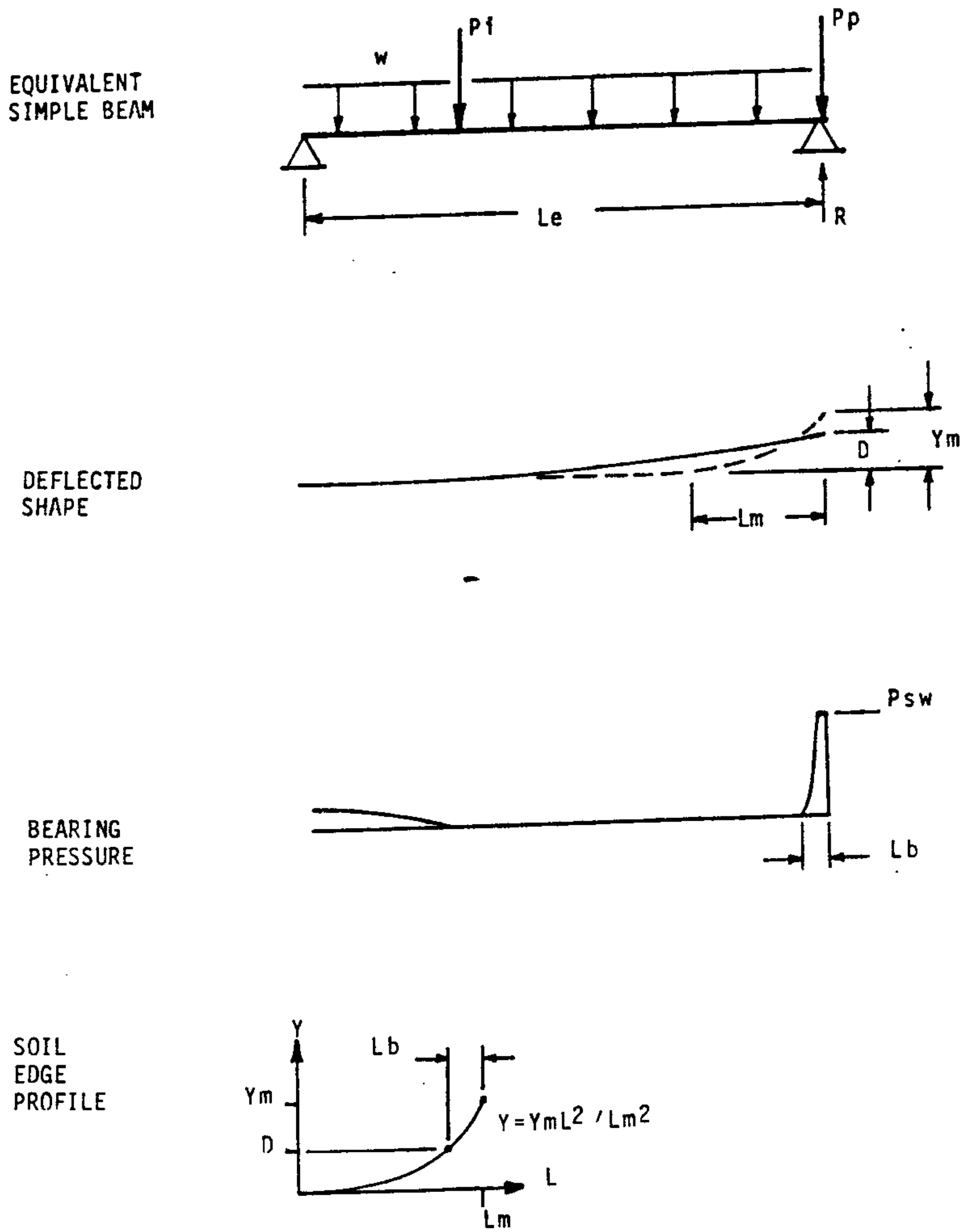


FIGURE A4 - EDGE LIFT DEFLECTION



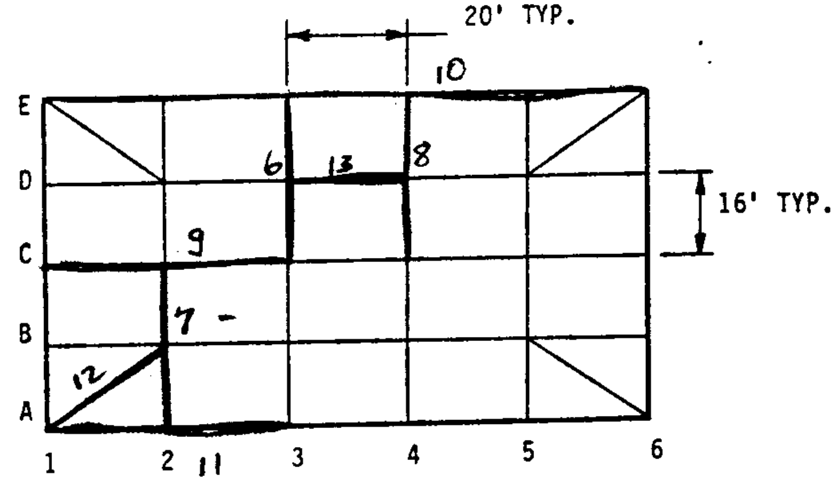
APPENDIX B - DESIGN EXAMPLE

(RIBBED MAT DESIGN IN EXPANSIVE SOIL)

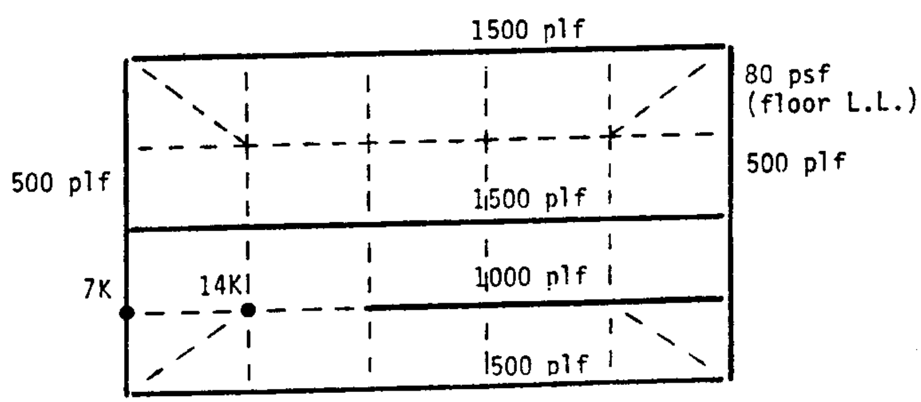
1. SOIL DATA (ref. Part I - 3.3)

qa = 2000 psf
Psw = (see page B9)
k = 100 pci
Lm = 6 ft
Ym = 1.5 in for center lift
Ym = 1.0 in for edge lift

2. FOUNDATION PLAN (ref. Part I - 4.3)



3. LOADS



4. BEARING DESIGN FOR RIBS (ref. Part I - 4.4)

Maximum wall load (P) = 1500 plf

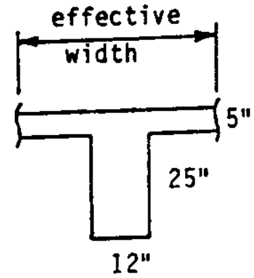
Width $\geq$ P/qa = 1500/2000 = .75 ft

Use 12 inch wide ribs (minimum)

5. INTERIOR RIB PROPERTIES (ref. Appendix A - 2.1)

Ec = 3,320,000 psi

(effective flange width per ACI 318, section 8.10.2 For "span length" use 4Lc for center lift or Le for edge lift)



Let Ir = 36,000 in^4 for center lift
Ir = 24,000 in^4 for edge lift
(ref. ACI 318, section 9.5.2.3, verify Ir after calculating M)

I = Ir/S (in^4/ft):

Rib spacing	16 ft	20 ft
Center lift Edge lift	2250 1500	1800 1200

6. CENTER LIFT DESIGN - RIB E3/C3

6.1 Loads (ref. Appendix A - 2.1)

slab weight = 150 pcf x 5/12 ft = 62 psf

w = DL + LL = 62 + 80 = 142 psf

rib weight = 150 pcf x 2.5 ft x 1.0 ft = 375 plf

Pp = rib + wall = 375 + 1500 = 1875 plf

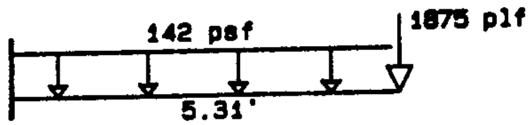
6.2 Equivalent cantilever (ref. Part II - 3.1)

$Lo = 2.3 + .4\ Lm = 2.3 + (.4 \times 6) = 4.7$ ft

$C = .8\ Ym^{.12}\ I^{.16} / Pp^{.12}$

$C = .8 \times 1.5^{.12} \times 1800^{.16} / 1875^{.12} = 1.13$

$Lc = Lo\ C = 4.7 \times 1.13 = 5.31$ ft



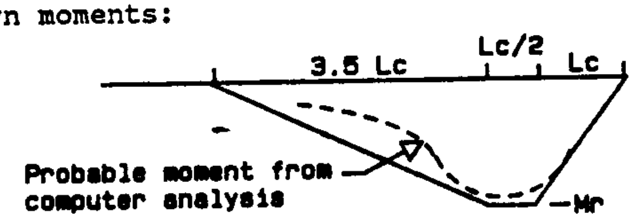
6.3 Moment (ref. Part II - 3.1.2)

$M = Pp\ Lc + 1/2\ w\ Lc^2$

$M = 1875 \times 5.31 + 1/2 \times 142 \times 5.31^2 = 12{,}000$ ft-lb/ft

$Mr = M \times S = 12000 \times 20 = 240{,}000$ ft-lb/rib

Design moments:

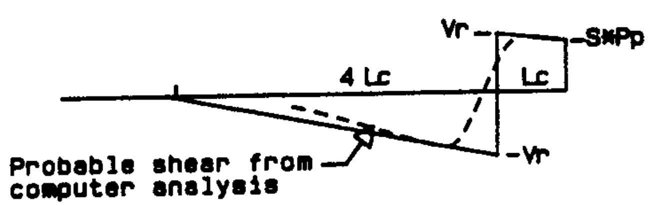


6.4 Shear (ref. Part II - 3.1.3)

$V = Pp + w\ Lc = 1875 + 142 \times 5.31 = 2630$ lb/ft

$Vr = V \times S = 2630 \times 20 = 52{,}600$ lb/rib

Design shears:



6.5 Reinforcing in rib (ref. Part I - 3.1.6 and 4.5)

$As = (Mr/ad)/1.33$

$As = 240 / (1.76 \times 28 \times 1.33) = 3.66 \text{ in}^2$ (top)

use 3 #10 bars

$v = Vr/bd = 52600 / (12 \times 28) = 157$ psi

$vc = (1.1\sqrt{f'c})1.33 = 80$ psi

$Av = (v-vc)b\ s/(fs\ 1.33)$

$Av = (157-80)\ 12 \times 12 / (24000 \times 1.33) = .35 \text{ in}^2/\text{ft}$

use #4 stirrups @ 12 in

6.6 Deflection (ref. Part II - 3.1.4)

$\theta = M^{1.4}/\ 9800\ I\ k^{.5}$

$\theta = 12000^{1.4}/\ (9800 \times 1800 \times 100^{.5}) = .0029$ radians

$D = .11 + 12\ Lc\ \theta = .11 + 12 \times 5.31 \times .0029 = .29$ in

$D/4Lc = .29 / (4 \times 5.31 \times 12) = 1/879$ O.K.

7. EDGE LIFT DESIGN - RIB A2/C2

7.1 Loads

$w = 142$ psf (same as above)

Pp = rib + wall = 375 + 500 = 875 plf

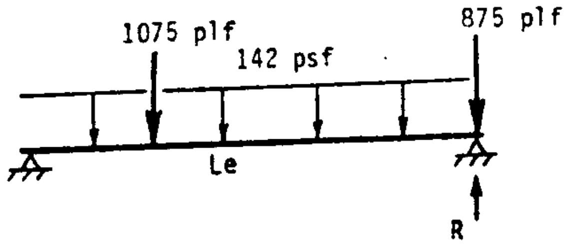
Pi = rib + wall* = 375 + 700 = 1075 plf

* equivalent wall load = column load / rib spacing

14000/20 = 700 plf (ref. Appendix A - 2.4)

$Li = 16$ ft

7.2 Equivalent simple beam (ref. Appendix A - 3.2.1)



7.3 Deflection (ref. Part II - 3.2.2)

$Le = 7.5\ I^{.17}\ Li^{.37}\ D^{.12} /\ w^{.07}\ Pi^{.11}$

$Le = 7.5 \times 1200^{.17} \times 16^{.37} \times D^{.12} /\ 142^{.07} \times 1075^{.11}$

$Le = 22.9\ D^{.12}$

assume D = .50 in (somewhat less than Ym = 1.0 in)

$Le = 22.9 \times .50^{.12} = 21.1$ ft

R = Pp + 1/2 w Le + Pi(Le-Li)/Le

R = 875 + (142x21.1)/2 + 1075(21.1-16.0)/21.1 = 2633 plf

from heave/pressure curve (p B9), for D=.50 find Psw=2000

Lb = 1.1(R/Psw) = 1.1(2633/2000) = 1.45 ft

$D = Ym(Lm-Lb)^2/Lm^2$

$D = 1.0(6.0-1.45)^2/6.0^2 = .575$ in $\neq$ .50 inch assumed!

assume D = .54 in

$Le = 22.9 \times .54^{.12} = 21.3$ ft

R = Pp + 1/2 w Le + Pi(Le-Li)/Le

R = 875 + (142x21.3)/2 + 1075(21.3-16.0)/21.3 = 2655 plf

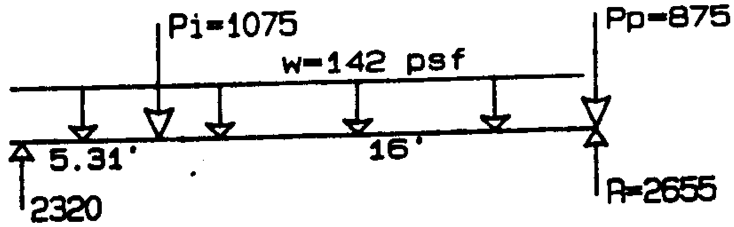
from heave/pressure curve, for D=.54 find Psw=1800 psf

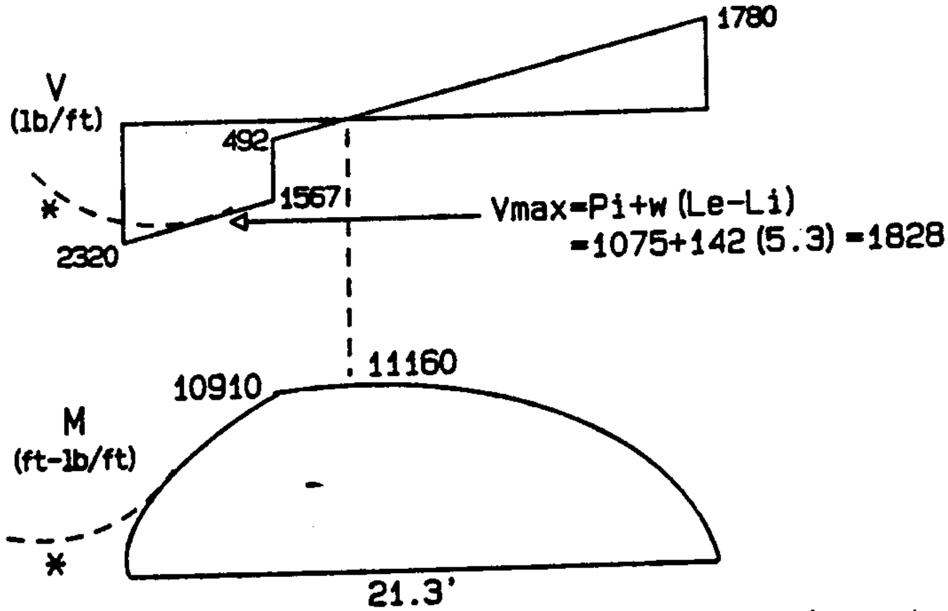
Lb = 1.1(R/Psw) = 1.1(2655/1800) = 1.62 ft

$D = 1.0(6.0-1.62)^2/6.0^2 = .533$ in CONVERGED!

D/Le = .54/(21.3x12) = 1/473 O.K. for non-brittle walls

7.4 Moment and shear (ref. Part II - 3.2.3 and 3.2.4)





* probable shear and moment from computer analysis, note that calculated V=2320 lb will not occur, due to the effects of distributed support from the soil

8. EDGE LIFT DESIGN - RIB E4/C4

8.1 Loads

w = 142 psf (same as above)

Pp = 1875 plf (same as rib E3/C3)

Li = 32 ft (wall along rib C1/C6)

8.2 Deflection

since Li>Le use:

$Le = 10.5\ I^{.17}\ D^{.12} / \ w^{.07}$ (ref. Part II - 3.2.5)

$Le = 10.5 x 1200^{.17} x D^{.12} / \ 142^{.07} = 24.77\ D^{.12}$

assume D = .48 in

then $Le = 24.77 \ x \ .48^{.12} = 22.7$ ft

R = Pp + 1/2 w Le = 1875 + (142x22.7)/2 = 3485 plf

from heave/pressure curve, for D=.48 find Psw=2100 psf

Lb = 1.1(R/Psw) = 1.1(3485/2100) = 1.825 ft

$D = Ym(Lm-Lb)^2/Lm^2$

$D = 1.0(6.0-1.825)^2/6.0^2 = .484$ inch CONVERGED!

8.3 Find shears and moments by statics, similar to rib A2/C2.

9. CENTER LIFT DESIGN - RIB C1/C3

9.1 Loads

w = slab + LL + wall* = 62 + 80 + 94 = 236 psf

* wall = wall load / rib spacing = 1500/16 = 94 psf
(ref. Appendix A - 2.4)

Pp = rib + wall = 375 + 500 = 875 plf

9.2 Equivalent cantilever

Lo = 2.3 + .4 Lm = 2.3 + (.4 x 6) = 4.7 ft

$C = .8\ Ym^{.12}\ I^{.16} / \ Pp^{.12}$

$C = .8 \ x \ 1.5^{.12} \ x \ 2250^{.16} / \ 875^{.12} = 1.28$

Lc = Lo C = 4.7 x 1.28 = 6.02 ft

9.3 Moment

$M = Pp\ Lc + 1/2\ w\ Lc^2$

$M = 875 \ x \ 6.02 + (236 \ x \ 6.02^2)/2 = 9544$ ft-lb/ft

Mr = M x S = 9544 x 16 = 153,000 ft-lb/rib

9.4 Shear

$V = Pp + w\ Lc = 875 + (236 \times 6.02) = 2296$ plf

$Vr = V \times S = 2296 \times 16 = 36{,}700$ lb/rib

9.5 Deflection

$\theta = M^{1.4} / 9800\ I\ k^{.5}$

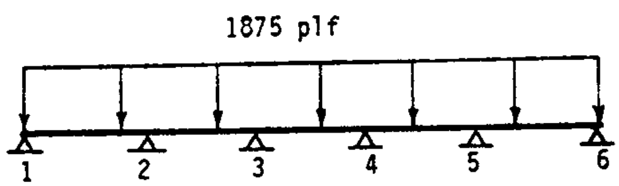
$\theta = 9544^{1.4} / 9800 \times 2250 \times 100^{.5} = .0017$ radian

$D = .11 + 12\ Lc\ \theta = .11 + (12 \times 6.02 \times .0017) = .23$ in

10. CENTER LIFT DESIGN - PERIMETER RIB E1/E6 (ref. Part II-3.3.1)

10.1 Span between transverse ribs

Pp = 1875 plf (from calculations for rib E3/C3)

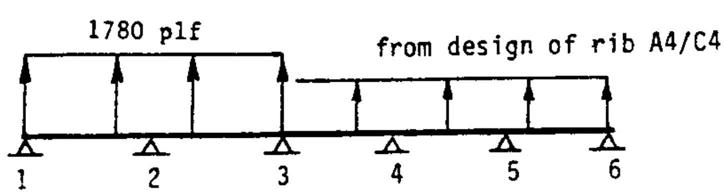


10.2 Analyze by conventional methods

11. EDGE LIFT DESIGN - PERIMETER RIB A1/A3 (ref. Part II - 3.3.2)

11.1 Span between transverse ribs for net upward force (from calculations on rib A2/C2)

R - Pp = 2655 - 875 = 1780 plf (upward)



11.2 Analyze by conventional methods

12. CENTER LIFT DESIGN - DIAGONAL RIB A1/B2 (ref. Part II - 3.4)

12.1 Provide the larger shear and moment capacity of rib B1/B2 or rib A2/B2.

13. RIB D3/D4 (ref. Part I - 4.5)

13.1 Interior rib with no wall or column loads

$A_s \geq .0033\ A_g = .0033 \times 12 \times 30 = 1.20\ in^2$ (top and bot.)

This is the typical minimum reinforcement for the full length of all ribs.

14. HEAVE VERSUS SWELL PRESSURE CURVE (ref. Appendix A - 2.1)