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

REPLY TO
ATTENTION OF

16 APR 1987

SWDED-G

SUBJECT: Criteria for Developing Geotechnical Design Parameter for SWD Ribbed Mat Design Methodology

Commander, Albuquerque District, ATTN: SWAED-TA
Commander, Fort Worth District, ATTN: SWFED-F
Commander, Galveston District, ATTN: SWGED-G
Commander, Little Rock District, ATTN: SWLED-G
Commander, Tulsa District, ATTN: SWTED-G

1. Reference is made to criteria letter SWDED-TS/G dated 23 December 1986, subject "Design Criteria for Ribbed Mat Foundation".

2. The above reference criteria letter require certain geotechnical parameters be furnished in the Foundation Design Analysis when a ribbed mat slab foundation is recommended in expansive soil areas. Enclosure 1, for addressees only, provides guidance for development of these parameters. These procedures were developed by the Ft. Worth District with review in the Southwestern Division. Questions and/or comments should be directed to either Mr. A.L. Branch, FTS 334-2117 or Mr. Jack Fletcher, FTS 729-6365.

FOR THE COMMANDER:

William Fisher

Encl

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

CONTENTS

DEVELOPMENT OF GEOTECHNICAL DESIGN PARAMETERS FOR RIBBED MAT FOUNDATIONS

1. REFERENCE.

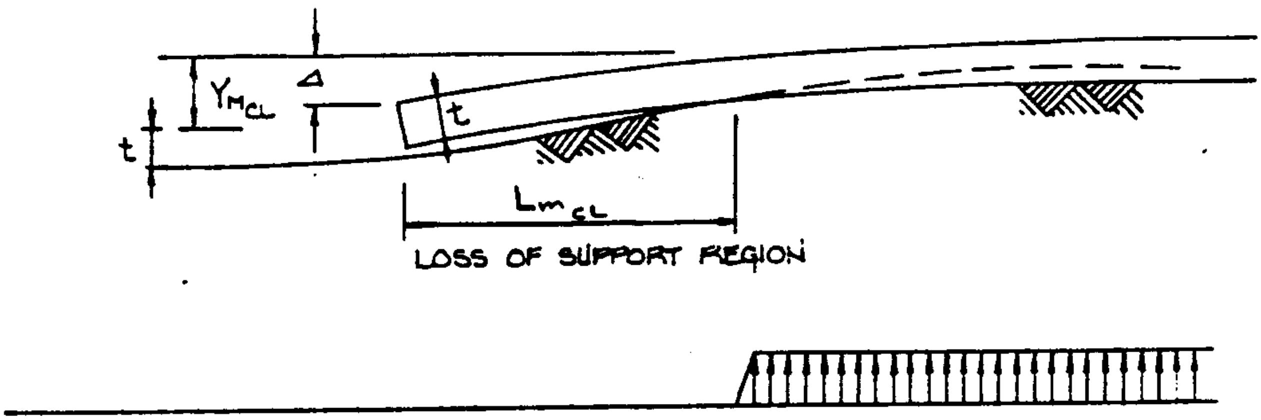
1.1 SWDED-TS/G, Design Criteria for Ribbed Mat Foundations, dated 23 Dec 86.

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

2. BACKGROUND. The recently developed structural design methodology (reference 1) models the interaction of a ribbed mat slab on an expansive subgrade for purposes of structural design. This method appears equally suited to stiffened mat systems such as flat mats, modified flat mats and inverted ribbed mats. Utilization of the methodology requires the expansion and refinement of the geotechnical design parameters furnished in the foundation design analysis. The purpose of this report is to (1) identify and (2) provide a rational method of determining these parameters.

3. SOIL-STRUCTURE INTERATION MODES. Two heave induced deformation conditions appropriate for ribbed mat slab structural analysis are (a) center lift and (b) edge lift.

3.1 CENTER LIFT. Center lift considers doming of the foundation in the interior region of a slab on grade differentially to the perimeter region as depicted on figure 1. This may be caused either by drying of the expansive subgrade around the perimeter beam or by wetting of the dry expansive subgrade in the interior region. Perimeter drying results from (1) below average precipitation and/or (2) reduced or no landscape watering and/or

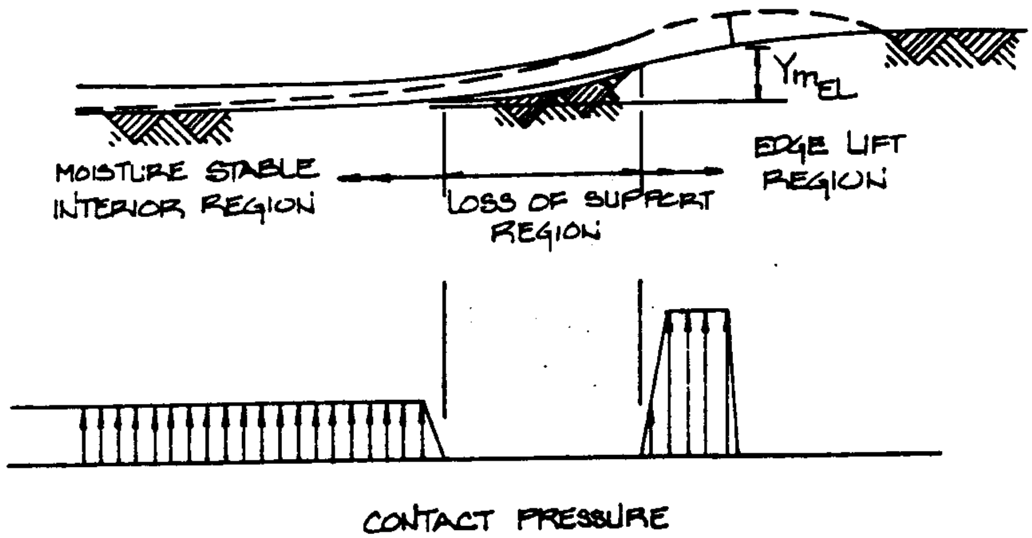


CENTER LIFT

FIGURE 1

(3) removal of old paving or hard stand. Interior wetting results from (1) disruption of the site moisture equilibrium by "capping" the site with the relatively impervious slab or by removal of thick brush or trees from the site (thus eliminating evapo-transportation) and/or (2) leaky inservice or abandoned utilities. Loss of support along perimeter and first interior transverse stiffener beam results if (1) the magnitude of center lift heave is large enough and (2) the beams are sufficiently rigid to cantilever from the supported interior region.

3.2 <u>EDGE LIFT</u>. Edge lift involves more complex soil-structure interactions than does center lift. In edge lift, the structure is supported by heaving subgrade in the perimeter region and in the relatively moisture stable interior region. Loss of support develops when (1) the edge lift heave deformation is large enough and (2) the spanning beam is sufficiently rigid. Edge lift mode is depicted on figure 2.



EDGE LIFT
FIGURE 2

Soil-structure interaction within the interior supported region is reasonably represented as a beam on non-linear subgrade. Soil-structure interaction in the perimeter region is somewhat more complex because the soil deflects under the structural load as a beam on non-linear subgrade, but also the swelling soil either loads and/or deflects the beam upward. To further complicate matters, the amount of edge lift heave and the soil-beam interface pressure are interrelated and unique for each specific site. Background parameter studies for reference 1 indicate that the structural analyses are particularly sensitive to edge lift parameters (edge lift heave magnitude and limiting beam-soil interface pressure). For example, large values for these may cause the solution to either fail to converge or indicate that the beam must be very deep and/or very heavily reinforced. While site conditions may sometimes dictate massive, very rigid stiffener beams, this is not generally the case. Generally, edge lift heave of less than 1.0 to 1.5 inches used in the design method given in reference 1 produce reasonable, constructable beams.

4. <u>DETERMINATION OF CENTER LIFT AND EDGE LIFT PARAMETERS FOR STRUCTURAL DESIGN</u>.

4.1 <u>CENTER LIFT</u> - Center lift parameters to be provided in the foundation design analysis includes (1) modulus of subgrade reaction (K_1), (2) design allowable bearing for beams (q_{all}), (3) magnitude of center lift (Y_{mCL}) and (4) loss of support distance around the perimeter (L_{mCL}).

4.1.1 <u>MODULUS OF SUBGRADE REACTION</u> - The modulus of subgrade reaction should be taken as K_1 = 200 pci for beams up to 12 inches wide bearing on compacted, nonexpansive fill. Higher values may be justified for granular nonexpansive fills consisting of gravel, crushed rock or limestone screenings or for cement stabilized materials if these materials extend significantly ($D \geq 3B$) below the stiffener beam of width B. The foundation design analysis should direct that K_1 values be factored to account for width effects such that $K_{design} = K_1/B$, where B is the effective beam width in feet for soil structure interaction. Note that the resultant effective beam width may include a significant width of the slab and is therefore significantly greater than actual beam width. Studies indicate that significant load distribution occurs over an "effective" width of approximately five. It should be noted that structural design calculations are not sensitive to K value.

4.1.2 DESIGN ALLOWABLE BEARING. A design allowable bearing value (q_{all}) has historically been assigned for sizing of stiffener beams, perimeter beams and enlarged beam intersections beneath columns. Values are typically given considering the beam to be a continuous strip footing or the beam intersection to be a spot footing (carrying either line or concentrated loads, respectively). The allowable bearing value is typically developed based on the average strength of engineered fill at shallow depth with a factor of safety of not less than 3.0. Design loads typically include full dead load plus half live load. The purpose in sizing the beams and beam intersections for this design allowable is to provide uniform contact pressures at the beam-soil interface therefore limiting inservice differential settlement. The assumptions of minimal load sharing between the slab and beams, ample safety factor on the fill strength, and minimum beam widths specified in the SWD EIM combine to limit the mobilized soil strains to low levels. This leads to very small structurally induced deflections given uniform, nominal fill depths. Actual values assigned for design bearing allowables have seldom exceeded q_{all} = 2.0 KSF although values as high as 3.0 KSF have been assigned in limited cases where required and justifiable. Seldom are there structural requirements for larger allowables bearing values since specified minimum beam widths generally govern.

4.1.3 MAGNITUDE OF CENTER LIFT HEAVE POTENTIAL. - The magnitude of center lift heave potential (Y_{MCL}) given in the foundation design analysis should be the residual heave potential at the site. The value of Y_{MCL} should include effects due to subgrade removal and replacement criteria, any surcharge effects due to fill above original subgrade and the weight of the proposed structure. Maximum design value for center lift potential should not exceed 1.5 inches. Where attainable with reasonable removal/replacement depths (≤36 inches), it is desirable to limit Y_{MCL} to not more than 1.0 inch, which is well within the "tolerable" inservice deformation range of most structures. Minimum remove/replace depth should be taken to the bottom elevation of the ribbed mat slab beams. function of Anticipated loads

The heave potential is determined by three soil parameters: the coefficient of swell (C_s), depth of active zone (X_a) and expansion pressure (P_{exp}).

Caution should be used in selecting coefficient of swell (C_s) values for heave analyses since swell pressure test results significantly underestimate C_s values compared to controlled expansion-consolidation-rebound tests. Additionally, both test methods tend to give low C_s values since most rebound time curves are terminated well before primary swell is completed.

★ An appropriate design value of the depth of the active zone (X_a) typically lies between the present depth to the stable relative moisture content (estimated by observing the relationship of moisture content to the plastic limit) and the maximum depth observed, such as the maximum depth of weathering. Typical X_a values for the central and north Texas regions and central Oklahoma region appear to vary from about 10 to 15 feet. These values have been estimated for (1) regression heave analyses for distressed structures and (2) depth of moisture variation versus approximate return/duration interval studies. Values smaller than 14 feet may be applicable in specific cases such as where the active zone is the distance between the structural foundation element or slab on grade and a perched water table; a condition common in these regions. *

Center lift heave analyses should consider "saturated" conditions to a depth of X_a. If a nominal remove/replace depth and saturated subgrade assumptions indicate unreasonable residual heave potential, consider increasing the depth of remove/replace and/or recommending a more defensive design to prevent saturation of the subgrade.

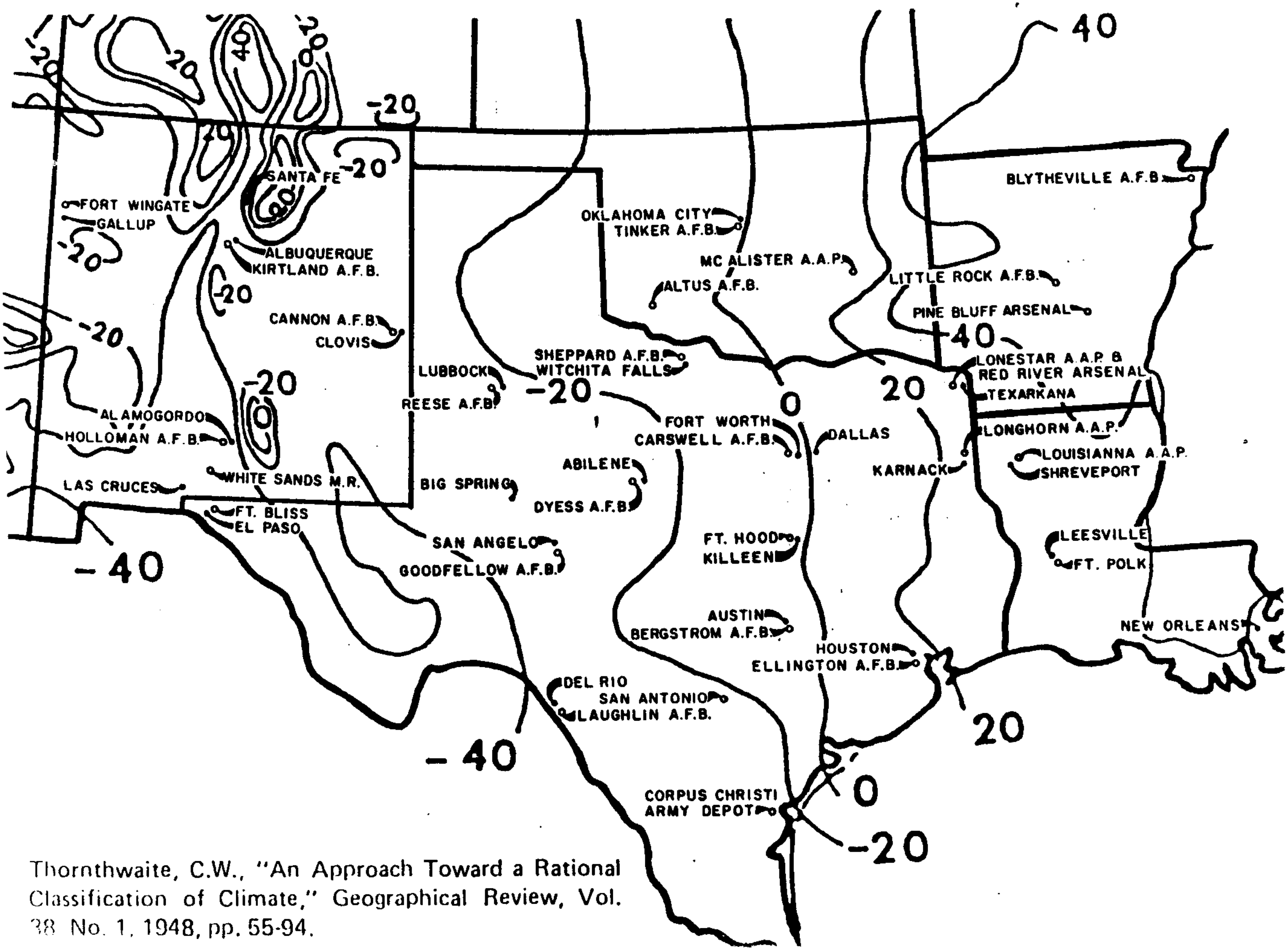
Expansion pressures should be developed versus depth using small depth intervals. These should be developed from laboratory data for the site. Additionally, these may be supplemented using proper correlations with nearby, preferably adjacent sites.

4.1.4 <u>EDGE MOISTURE VARIATION DISTANCE</u>. The edge moisture variation distance (L_{MCL}) may control the design of interior stiffener beams which are adjacent to the perimeter. The maximum moments and shear are induced in the transverse beams when these elements cantilever free of foundation support from the interior supported region to the outside of the perimeter beam. The length of cantilver is largely controlled by the value of L_{MCL}. SWD adopted this concept from Post-Tensioning Institute (PTI) guidelines, originally developed for lightly loaded flexible mats in the late 1970's and early 1980's. Standard practice in the

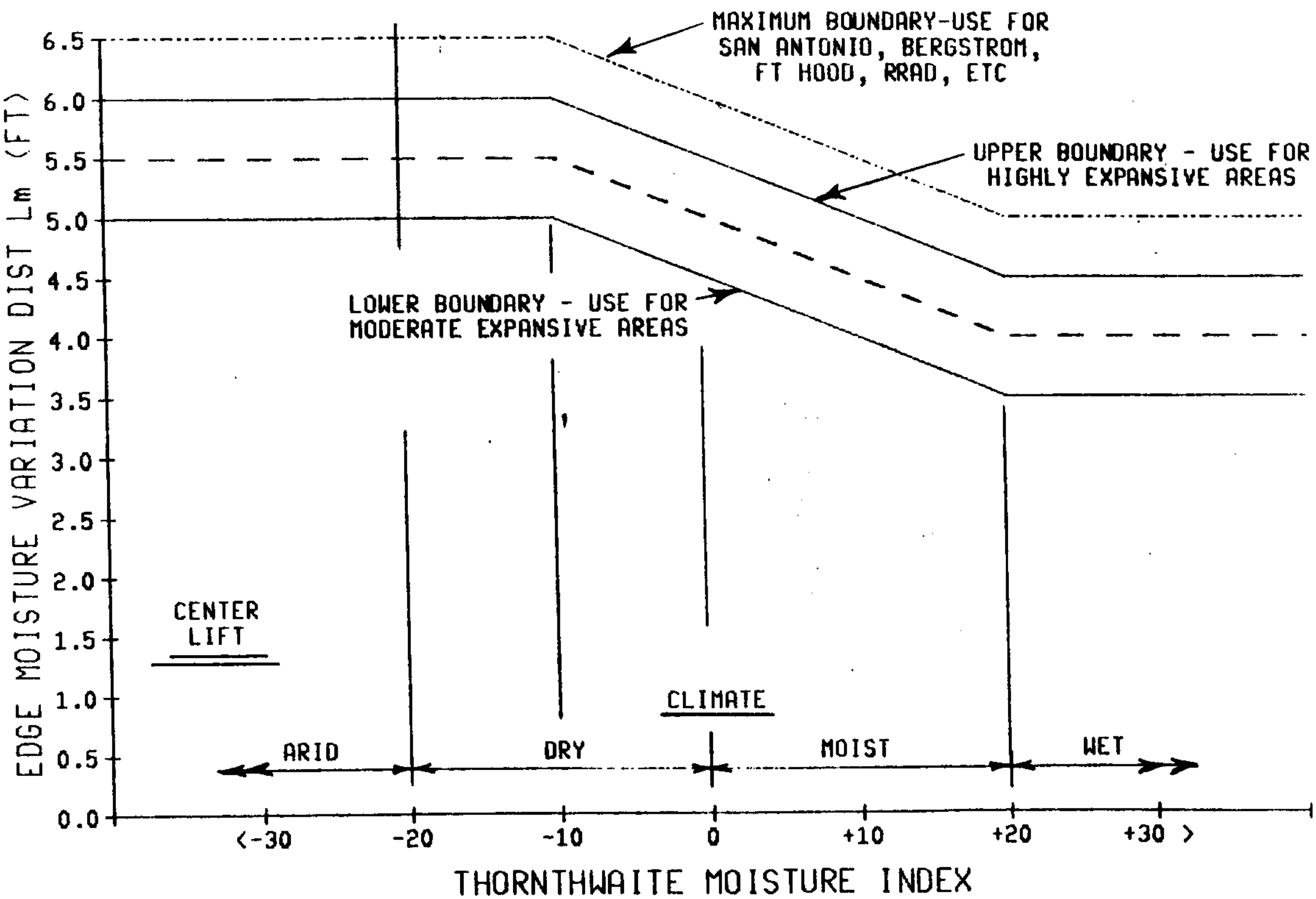
San Antonio area has been to assign upper or near upper bound values from TMI for design L_{mcL} values. At least two aspects of designs probably tend to moderate the actual edge moisture variation distance experienced; these being (1) relatively deep perimeter beams which act as a physical barrier and (2) the non-expansive fill blanket which tends to make changes in moisture content (and therefore any resultant heave or shrinkage) more uniform and provide a surcharge effect as well. Other factors, however, tend to offset these moderating effects. These include very short return interval of edge moisture variation events presented in TMI (reported by some sources to range from 1 to 2 years). Typical project design life of projects exceeds 20 or 30 years and, since we're still using many World War II facilities, it may well exceed 50 years. Estimated edge moisture variation values considering a 100 percent probability of experiencing a 20 to 30-year return interval event may well be twice typical TMI values.

Based on a subjective combination of all factors, it is suggested that L_{mcL} be taken as the edge moisture variation distance determined using figures 3 and 4. These values should be modified, either up or down, based on site secific soils investigations and engineering judgement.

4.2 <u>EDGE LIFT</u> - Edge lift parameters to be provided in the foundation design analysis include (1) modulus of subgrade reaction (K_1), (2) magnitude of edge lift heave (Y_{meL}), (3) limiting soil-beam interface pressure (P_{sw}) for that portion of the beam being acted on by the heaving subgrade and (4) a value for edge moisture variation distance (L_{meL}).



Thornthwaite, C.W., "An Approach Toward a Rational Classification of Climate," Geographical Review, Vol. 38 No. 1, 1948, pp. 55-94.



APPROXIMATE RELATIONSHIP BETWEEN THORNWAITE INDEX AND MOISTURE VARIATION DISTANCE

FIGURE 4

4.2.1 MODULUS OF SUBGRADE REACTION. - Values given for center lift are considered appropriate for edge lift also. $K_1 = 200$ pci

4.2.2 SOIL-BEAM INTERFACE PRESSURE. Discussion of both limiting soil-beam interface pressure and magnitude of edge lift heave parameters (P_{SW} and Y_{MEL}) are best handled concurrently since both are intimately related and the analysis necessary for solution determines both simultaneously.

The area of soil-beam contact in the swelling perimeter region involves a somewhat complex soil-structure interaction situation. As edge lift develops and loss of support occurs between the perimeter and interior regions, the heaving soil may well exert a pressure on the stiffener beams well in excess of typical design interface pressures (q_{all}). As the soil column swells and lifts the overlying beam, the soil-beam contact area increases toward the interior region to accommodate the greater structural reaction.

The soil-structure interaction in the edge lift region can be visualized as a three-component system; (1) a structural element (a beam or mat strip), (2) an element of nonexpansive fill beneath the structural element plus that piece of the expansive subgrade restrained against heave by the weight of the overlying fill and the stresses induced beneath the structural element, and (3) the heaving column of soil to a depth of X_a beneath the bottom of the nonexpansive fill blanket (figure 5).

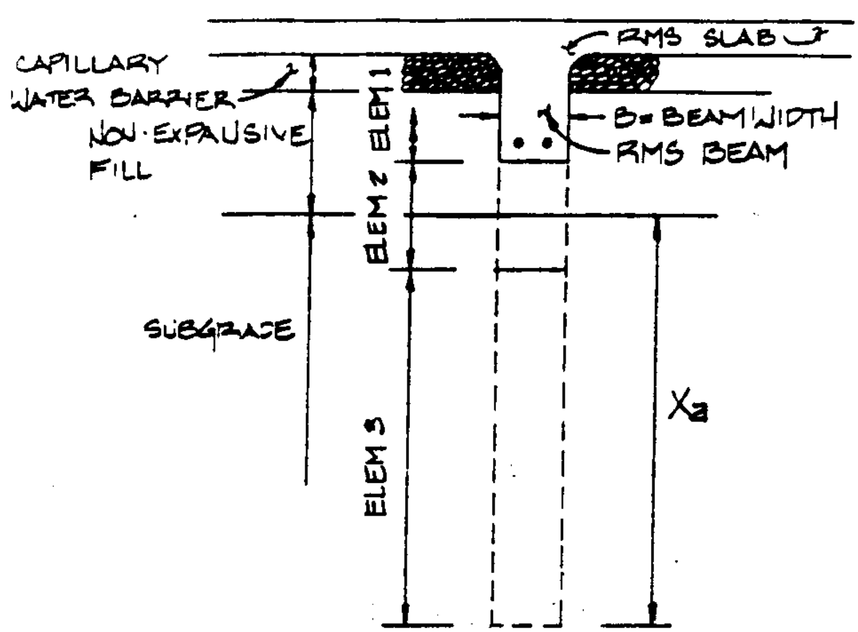


FIGURE 3

The load-deformation relationship of element 1 interacting with element 2 can be represented by a P-Y curve shown in figure 6.

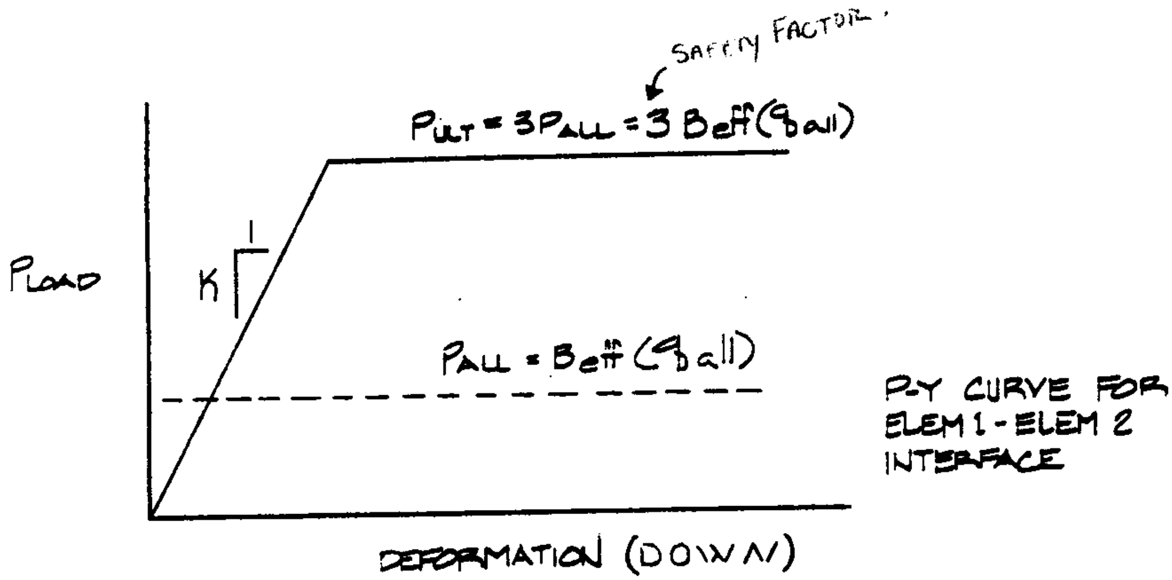


FIGURE 6

The load-deformation relationship of element 3 interacting with elements 1 and 2 in the column immediately below the beam as shown on figure 7. The plot consists of the net heave potential of the swelling soil column versus those forces resisting the tendency to swell, taken at the base of the structural beam.

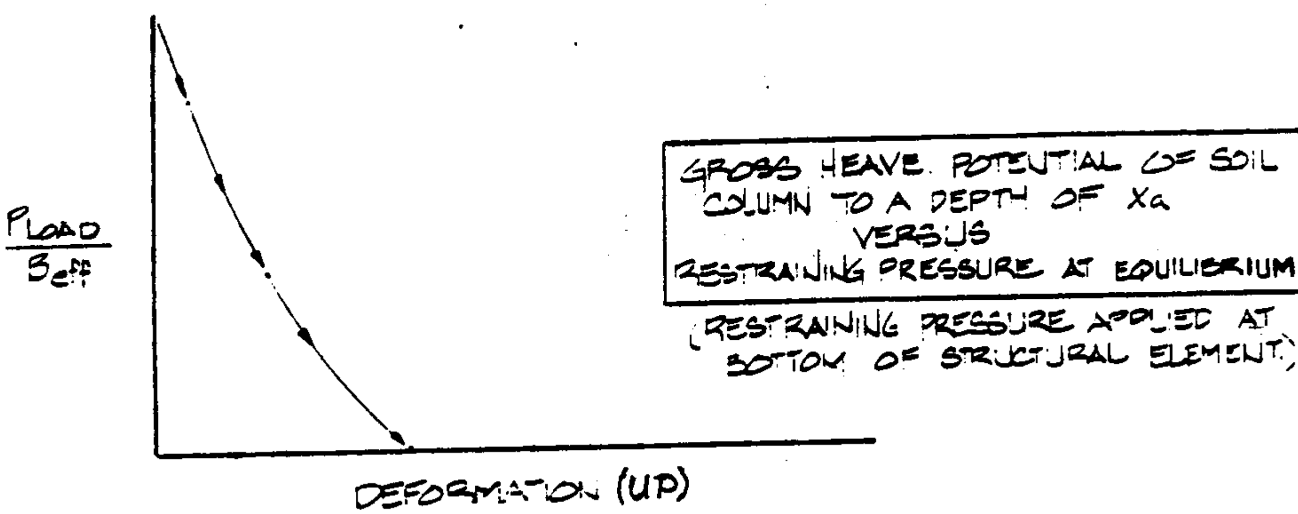


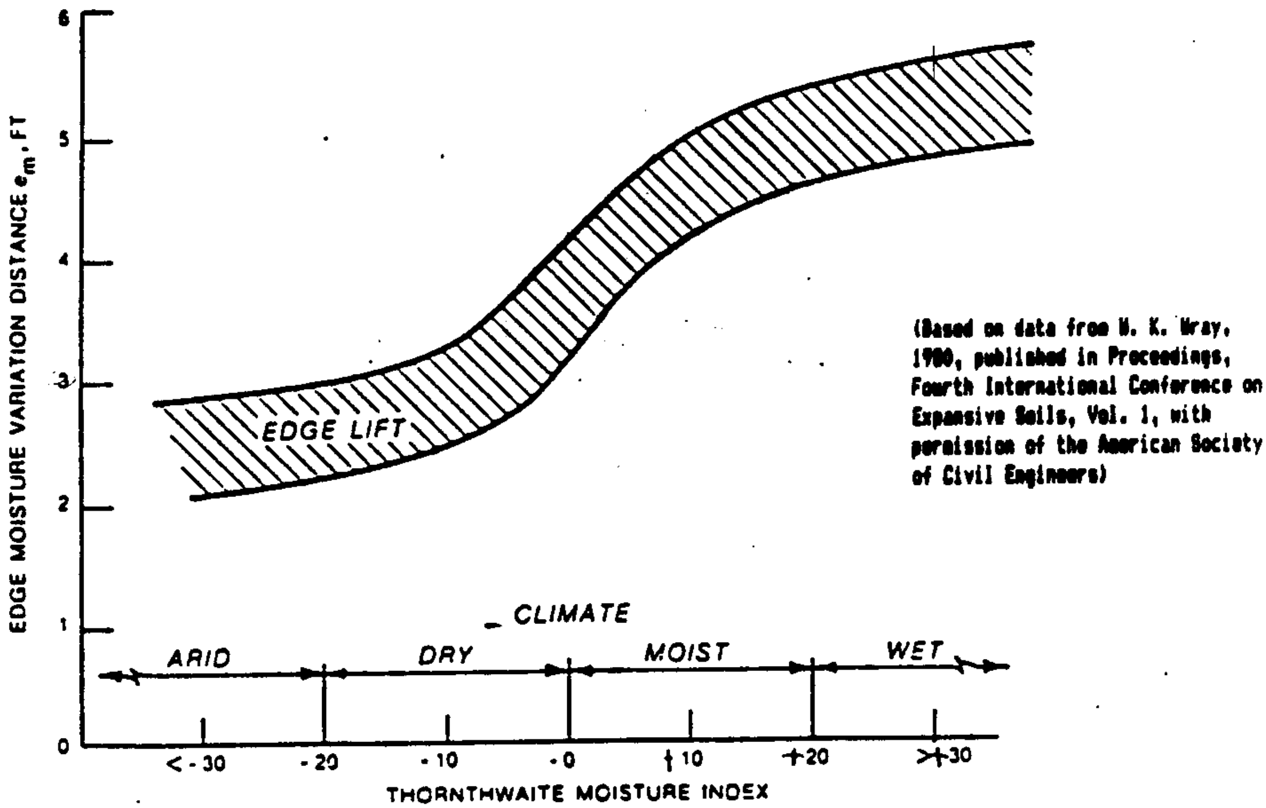
FIGURE 7

These relationships can be added algebracially to produce a composite p-y curve which can be easily utilized by available soil-structure interaction programs for structural analysis. Since such analysis is within the purview of the structural engineer, the geotechnical engineer need only furnish the pressure heave relationship in useable form in the Foundation Design Analysis. It is suggested that this information be provided in a tabulated format giving coordinates for at least three points. These minimum three points should be the P_{SW} and

P_{ULT} = F.S. x P_{ALLOW}

Y_{MBL} coordinates for (1) pressure equal to P_{ult}, (2) pressure equal to P_{all} and (3) pressure equal to zero.

4.2.3 EDGE MOISTURE VARIATION DISTANCE. Edge moisture variation distance (LmEL) appropriate for edge lift analysis may be taken from the TMI chart given in figure 8.



Approximate relationship between the Thornthwaite Moisture Index and the edge lift-off distance.

FIGURE 8

The TMI values represent approximate environmentally induced events. As a result, upper bound values should be selected for design. It is recommended, however, that average values be used for all SWD projects. Additionally, recommendations should be made in the foundation design analysis to limit the potential for developing "hot spots" due to long term sources of free water around the building perimeter.

function of the type of bldg, ie, Brick Stucco or Not

4.3 Certain structure-site situations may well warrant deleting edge lift analyses as follows:

4.3.1 Where the proposed structure is a pre-engineered metal building <u>without</u> interior masonry walls or heavy interior dead or permanent live loads.

4.3.2 Where defensive design efforts have been incorporated and reasonable confidence exists that these will be constructed and maintained as intended.

4.3.3 Where minor architectural distress (such as cracking of masonry walls, plaster walls, tiled surfaces) is not likely to cause undue user concern or raise inservice maintenance requirements significantly.

5. <u>APPENDIX A</u>

5.1 <u>EXAMPLE PROBLEM</u>. An example problem is provided in Appendix A.

APPENDIX A

EXAMPLE PROBLEM

1. Required. - Develop geotechnical parameters for the structural design of a ribbed mat slab given the following:

a. Proposed Structure. - Office/Administration type structure located in San Antonio, Texas, 60X150 feet in plan. The structure is to consist of double wythe masonry (face brick over CMU) load bearing exterior walls and isolated interior colums at 20 ft. centers.

b. Proposed Site. - One acre, minimal topographic relief, site covered with mesquite trees.

c. Subsurface Conditions. - Drilling program (5 borings) indicates the foundation materials consist of (1) a surface stratum of high plasticity clay grading into medium plasticity clay with depth to a total thickness of 14 feet, (2) a water bearing sand and gravel stratum from 1 to 7 feet thick overlying, (3) an expansive clay shale formation.

d. Summary Laboratory Test Data. -

Stratum	Depth (ft)	USCS	$\bar{W}_o$ (%)	d (pcf)	LL	PI	P_{exp} (tsf) (net)	C_s	C_c	C_u (ts
1	0-4	CH	25	105	65	45	0.8 -1.0	0.06	0.02	0.
2	4-14	CL	14	108	44	30	0.6	0.06	0.18	0.
3	14-20	GC	6	-	25	12	0	-	-	5C B/
4	20 plus	Wea. Clay Shale	22	110	70	52	2.0	0.09	0.22	1.

2. Determine Parameters Required for Center Lift Analysis:

a. Modulus of Subgrade Reaction (K_1). - Mat slab will be founded on nonexpansive fill, therefore it is reasonable to assign a valve of K_1 = 200 PCI. The structural engineer should factor this value based on effective beam width such that $K_{design} = K_1$ (1ft/Beff, ft).

b. Design Bearing Allowable (q_{all}). - Since beams will be supported on nonexpansive fill and the building loads will range from light to moderate, it appears that a design bearing allowable of q_{all} = 2.0 KSF is appropriate.

c. Magnitude of Center Lift Heave Potential (Y_{mcl}). -

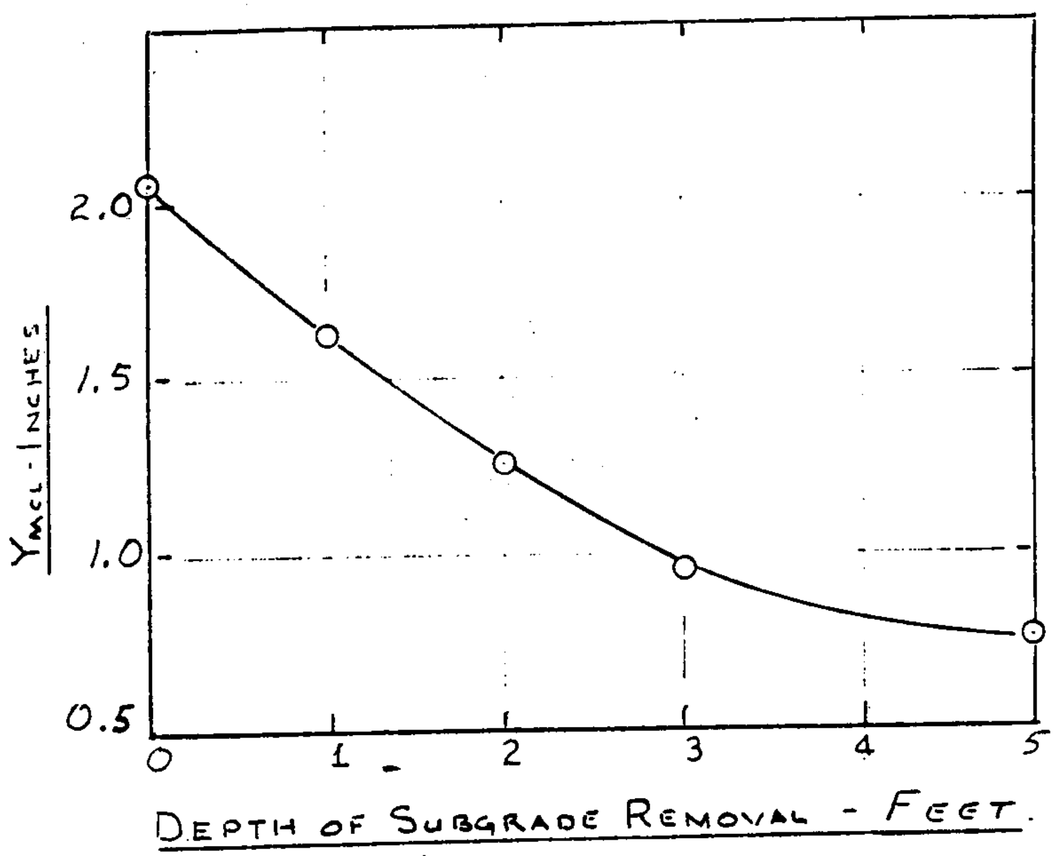
(a) Calculate site heave potential

Given: C_s = 0.06, e_o = 0.60, P_o = effective overburden pressure, P_{exp} = gross swell pressure, P_f = effective pressure resisting heave beneath mat including P_o, (surcharge due to fill and structural dead load, h_i = heave for soil layer h inches thick, and an X_a = 10 feet.

$C_s = .06$
$e_0 = .60$

z (ft)	z (ft)	P_o (tsf)	P_{exp} (tsf)	P_f (tsf)	h (in)	$h_i = \frac{C_s h}{1+e_o} \log_{10} \frac{P_{exp}}{P_f}$ (inches)	h_i (bottom to top) (inches)
0-1	0.5	0.03	1.0	.07+.03 0.1	12	0.45	2.07
1-2	1.5	0.1	1.0	.07+.1 0.17	12	0.35	1.62
2-3	2.5	0.17	1.1	.07+.17 0.24	12	0.30	1.27
3-4	3.5	0.23	0.8	0.3	12	0.19	0.97
4-6	5	0.33	0.9	0.4	24	0.31	0.78
6-8	7	0.46	1.0	0.53	24	0.25	0.47
8-10	9	0.6	1.2	0.67	24	0.22	0.22
10-12	11	0.73	1.35	0.8	24	0.2	N/A
12-14	13	0.86	0.9	0.93	24	N/A	N/A

Determine required depth of subgrade replacement and residual heave potential after replacement with nonexpansive fill. A plot of replacement depth versus residual heave taken from the above table follows:



Removal and replacement to 3.0 feet will reduce the heave potential to approximately 1.0 inch, thus Y_{mcl} = 1.0 inch. Note that significant additional removal would be required to reduce the residual heave potential any sigificant additional amount.

d. <u>Edge Moisture Variation Distance (L_{mcl})</u> - taken from figures 3 and 4 as L_{mcl} = 6.5 feet.

3. Determine parameters required for Edge Lift analyses:

a. <u>Modulus of Subgrade Reaction (K_1)</u>. - Same as for Center Lift.

b. <u>Design Allowable Bearing (q_{all})</u>. - Same as for Center Lift.

c. <u>Soil - Beam Interface Pressure (P_{sw})</u> and <u>Magnitude of Edge Lift Heave Potential (Y_{mel})</u>. -

Determine the residual heave potential for the soil column beneath a typical beam for a range of assumed interface pressures.

A summary of calulations and results is presented in tabulated form on page 6. A plot of soil-beam interface pressure versus heave potential is shown on page 7. A reasonable bilinear representation of the results, for use by the structural engineer, can be developed assuming a linear relationship between the following points:

P_{sw}, TSF	Y_{mel}, Inches
0.0	Y_{mel} = 1.25
q_{all} = 1.00	Y_{mel} = 1.0
q_{ult} = 3(q_{all}) = 3.00	Y_{mel} = 0.6

d. <u>Edge Moisture Variation Distance. (L_{mel})</u>. The edge moisture variation distance is taken from figure 8 as L_{mel} = 3.0 feet.

Given: D_f = 2.0ft B = 1.0ft

C_s = 0.06 e_o = 0.60

γ_m = 125pcf

								q_{app}											
								0.5tsf			1.0tsf			2.0tsf			4.0tsf		
Z ft	$\bar{Z}$ ft	h in	$\frac{\bar{Z}-D_f}{B}$	I_s	P_o tsf	P_o+P_s tsf	P_{exp} tsf	ΔP tsf	P_f tsf	Δh_i in	ΔP tsf	P_f tsf	Δh_i in	ΔP tsf	P_f tsf	Δh_i in	ΔP tsf	P_f tsf	Δh_i in
0-3	<----------------COMPACTED NONEXPANSIVE FILL TO A DEPTH OF 3.0 FT.---------------->																		
3-4	3.5	12	1.5B	0.28	0.23	0.30	0.8	0.14	0.37	0.15	0.28	0.51	0.08	0.56	0.79	0	1.12	1.35	0
4-6	5.0	24	3.0B	0.15	0.33	0.40	0.9	0.07	0.40	0.32	0.15	0.48	0.25	0.30	0.63	0.14	0.60	0.93	0
6-8	7.0	24	5.0B	0.09	0.46	0.53	1.0	0.05	0.53	0.25	0.09	0.55	0.23	0.18	0.64	0.17	0.36	0.82	0.09
8-10	9.0	24	7.0B	0.07	0.60	0.67	1.2	0.04	0.67	0.23	0.07	0.67	0.23	0.14	0.74	0.19	0.28	0.88	0.12
10-12	11.0	24	9.0B	0.05	0.73	0.80	1.35	0.03	0.80	0.20	0.05	0.80	0.20	0.1	0.83	0.2	0.20	0.93	0.16
12-14	13.0	24	11.0B	0.04	0.83	0.90	0.9	0.02	0.90	0.0	0.04	0.90	0.0	0.08	0.91	0	0.16	0.99	0
								$\Sigma \Delta h_i$ = 1.15			$\Sigma \Delta h_i$ = 0.99			$\Sigma \Delta h_i$ = 0.7			$\Sigma \Delta h_i$ = 0.38		

Where:

Z = depth interval

$\bar{Z}$ = mean depth

B = beam width

D_f = beam depth

σ_z = stress with depth

P_o = overburden pressure

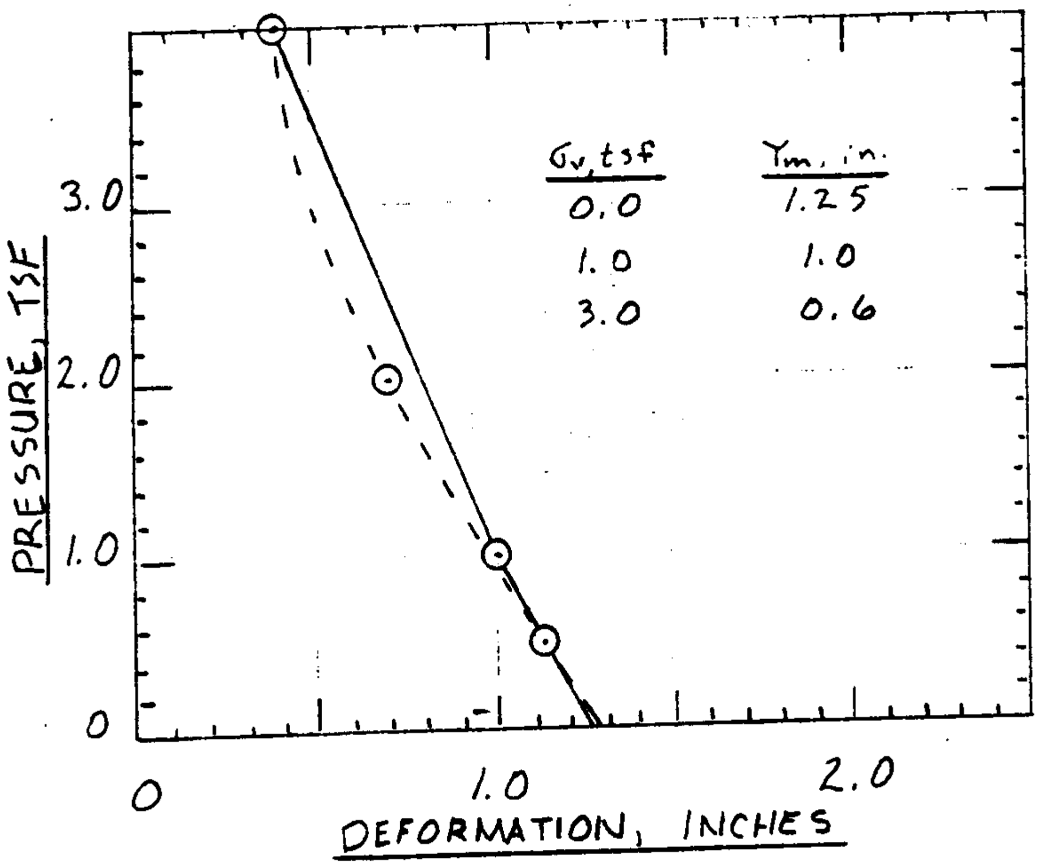
P_s = surcharge pressure next to beam

P_{exp} = expansion pressure

P_f = vertical pressure resisting heave below beam

P = stress @ depth due to to q_{app} $\Delta P = (I_s)(q_{app})$

$$P_f = \begin{cases} \Delta P + P_o \\ \text{or} \\ P_o + P_s \end{cases} \Big| \text{ whichever is greater}$$



EDGE LIFT PARAMETER

EXAMPLE PROBLEM