APPENDIX D

ENGINEERING CALCULATIONS ANALYZING THE POTENTIAL FOR IMPACTS ON LANDFILL SECTION 6/7 SOIL-BENTONITE CUTOFF WALL

Geosyntec consultants

COMPUTATION COVER SHEET

Client: FO **Project:** Fresh Kills **Project #:** ME0530B **Task #:**

TITLE OF COMPUTATIONS **FINITE ELEMENT ANALYSIS FOR SLURRY WALL**

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FINITE ELEMENT ANALYSIS FOR SLURRY WALL

1 PURPOSE

The alternate design of Section 6/7 Final Cover System is being prepared in a manner that will allow it to support potential future roadways. The proposed service roads alignments will cross over or pass adjacent to the perimeter slurry cut-off walls. The purpose of this calculation is to evaluate if excessive deformation will be induced on the slurry wall by the load associated with road construction and operation, i.e., pavement structure own weight and traffic loads.

2 CROSS SECTION ANALYZED

At the time of this calculation, design of the road is ongoing. Therefore, for this analysis, it is assumed that a typical section of the roads at the vicinity of the slurry wall comprise of the following layers, from top to bottom:

- 6 inch of asphalt pavement;
- 3 ft of granular subbase;
- Final cover geomembrane (not modeled);
- Refuse; and
- Quaternary recent clay (Qrc).

A concrete slab will be used to bridge the road over the slurry wall. The thickness and length of the concrete slab are to be determined in this calculation. Figure 1 shows an idealized cross section for this analysis.

3 DESIGN LOADING

The design traffic loading for the road is assumed to be exerted by AASHTO HS 20 truck, as shown in Appendix A.

The static loads applied on three axles are 8,000 lbs, 32,000 lbs, and 32, 000 lbs. This static load is increased by 20% to account for the dynamic effect. Tire contact area is assumed to be 10 in. (in the direction of traffic) by 20 in. (in the direction normal to traffic). The distance between the two tires on the same axle is considered to be 6 ft. To convert the 3-dimensional loading to the plain strain 2-dimensional condition, the uniformly distributed loading corresponding to each axle load is calculated as follows:

$$\frac{1.2 \times 32,000\, lbs}{6\, ft \times (10/12)\, ft} = 7,680\, lbs/ft^2$$ (for 32,000 lbs static axle load)

$$\frac{1.2 \times 8,000 lbs}{6 ft \times (10/12) ft} = 1,920 lbs/ft^2$$ (for 8,000 lbs static axle load)

Three different loading positions (denoted as Position A, B and C) were considered to study the effect on the slurry wall as the truck cross the slurry wall. The three loading positions are illustrated in Figure 2.

4 ANALYSIS METHODOLOGY

The analysis was conducted using PLAXIS® version 8.2, a 2-D finite element program [PLAXIS BV, 2006]. Plane strain condition was assumed in the analysis. PLAXIS discretizes the soil mass using 15-node triangular elements. Figures 3 shows the finite element model for the cross section analyzed. Different constitutive models were used to characterize the foundation soils, slurry walls, and refuse. A description of each of the models used is presented in the section below.

In order to model the sequence of construction, the analysis was performed in 5 phases, which are as follows:

- *Phase 0: Initial Conditions.* This phase calculates the initial stress state prior to slurry wall installation and pavement construction using the k_0 procedure, in which the horizontal stress is calculated as vertical stress multiplied by the static earth pressure coefficient (k_0).

- *Phase 1: Slurry Wall Installation.* In this phase, the soil clusters at the location of the slurry walls were converted into slurry wall clusters to simulate the construction of slurry wall.

- *Phase 2 and 3: Pre-loading.* Waste under potential crossing locations is considered to be degraded and has been subjected to loads from the work platform construction and building equipment traffic during slurry wall construction. In the Yukon Avenue Corridor, the proposed crossing location would coincide with existing service road crossing. To take in account loading and waste degradation effects, a surcharge of 2,400 psf is applied at surface ground in Phase 2. The applied surcharge is removed in Phase 3.

- *Phase 4 through 6: Traffic Loading at Positions A, B and C.* In these phase, the plate elements representing the pavement and the concrete slab are turned on. All deformations prior to traffic loading are reset to zero in these three phases. The three cases, where traffic loads are applied at Positions A, B and C, were calculated independently.

5 MATERIAL PARAMETERS

The foundation clay (Qrc) and waste have been characterized in the report entitled "Deformation and Stability Analysis for Slurry Trench Cutoff Wall, Section 1/9 and Section 6/7 [Woodward-Clyde, 1993]. The material parameters used in this analyses for these two materials were selected to be the same as values previously used. The input parameters for the granular subgrade, pavement and concrete slab were selected from typical values for these materials. A discussions on these material properties are presented below.

Foundation Clay (Qrc)

The stress-strain behavior of the foundation clay (Qrc) is assumed to be represented by the PLAXIS Hardening Soil Model. The PLAXIS Hardening-Soil model is an advanced model that simulates the behavior of soils by accounting for stress dependent stiffness moduli. It uses a hyperbolic relationship between the axial strain (ε) and the deviatory stress (q) during primary triaxial loading described by the following equation:

$$\varepsilon = \frac{1}{2 \cdot E_{50}} \cdot \frac{q}{1 - \frac{q}{q_a}} \qquad (1)$$

where: q = Deviatory stress;
q_a = Asymptotic (or ultimate) deviatory stress;
ε = Axial strain; and
E_{50} = Stiffness modulus at 50 percent of q_f (q at failure).

The variation of q_a with confining pressure is accounted for by relating q_a to the compressive strength or deviatory stress at failure (q_f). The relationship between q_a and q_f is defined by the following equation:

$$R_f = \frac{q_f}{q_a} \qquad (2)$$

where R_f is the failure ratio and is always less than one. Using PLAXIS Hardening-Soil Model, E_{50} is the stress dependent stiffness modulus for primary loading. The stress dependency of E_{50} on the confining pressure is defined by the following equation:

$$E_{50} = E_{50}^{ref} \cdot P^{m} \tag{3}$$

$$P = \left(\frac{c' \cdot \cos\phi' + \sigma_3' \cdot \sin\phi'}{c' \cdot \cos\phi' + p^{ref} \cdot \sin\phi'} \right) \tag{4}$$

and E_{50}^{ref} is a reference stiffness modulus corresponding to reference confining pressure p^{ref} (atmospheric pressure of 2,116 psf was used as p^{ref} in the calculation). σ_3' is the effective minor principal stress; and m is the modulus exponent that governs the stress dependency.

The inelastic behavior during unloading and reloading is represented by the stress dependent stiffness modulus for unloading and reloading (E_{ur}). The variation of E_{ur} with confining pressure is defined by the following equation:

$$E_{ur} = E_{ur}^{ref} \cdot P^{m} \tag{5}$$

where E_{ur}^{ref} is a reference stiffness modulus for unloading and reloading corresponding to reference confining pressure p^{ref} and P and m are as discussed above.

Similarly, the stress dependency of the tangent stiffness modulus for oedometer loading (E_{oed}) and unloading/reloading ($E_{oed,ur}$) are defined by the following equations:

$$E_{oed} = E_{oed}^{ref} \cdot P^{m} \tag{6}$$

$$E_{oed,ur} = E_{oed,ur}^{ref} \cdot P^{m} \tag{7}$$

where E_{oed}^{ref} and $E_{oed,ur}^{ref}$ are the reference stiffness modulus for oedometer loading and unloading/reloading corresponding to reference loading p^{ref}. E_{oed}^{ref} and $E_{oed,ur}^{ref}$ can be obtained from the consolidation parameters (i.e., compression index (C_c) and recompression index (C_r)). By default, PLAXIS assumes that E_{50}^{ref} =1.25 E_{oed}^{ref}.

For the foundation clay (Qrc) at the site, the power, m, is assumed to be 0.85, and the effective shear strength envelope of this material is assumed to be defined by a friction angle of 36 degrees and zero cohesion, as previously used by Woodward-Clyde[1993]. E_{50}^{ref}, E_{oed}^{ref} and E_{ur}^{ref} were calculated from parameters previously used by Woodward-Clyde[1993]. The calculations of these parameters are included as Appendix A.

Refuse

The stress-strain response for the refuse is assumed to be represented by the PLAXIS Hardening Soil Model, the same model used for Qrc. The moist and dry unit weight of the refuse is assumed to be 73 pcf.

The effective friction angle (ϕ) is assumed to be 28° , and cohesion (c') is assumed to be 300 psf, based on the values used by Woodward-Clyde [1993]. The calculation of E_{50}^{ref} , E_{oed}^{ref} and E_{ur}^{ref} are also included in Appendix B.

Slurry Wall

The slurry wall material is assumed to be a soil-bentonite backfill. The stress-strain response for the slurry wall is also assumed to be represented by the PLAXIS Hardening Soil Model.

The unit weight of the slurry wall is assumed to be 105 pcf and the cohesion is assumed to be 20 psf based on the value used by Woodward-Clyde [1993] based on laboratory test results. Although the material is assumed frictionless, a nominal friction angle of 0.5 degree is assumed to avoid numerical instability during modeling. The deformation characteristics E_{50}^{ref} , E_{oed}^{ref} and E_{ur}^{ref} are derived from previous parameters developed by Woodward-Clyde, as shown in Appendix B.

Granular Subbase

The subbase of the road is assumed to be a granular material. The stress-strain response for the granular subbase is also assumed to be represented by the PLAXIS Hardening Soil Model. The effective friction angle (ϕ) is assumed to be 34° . Although the granular subbase is considered to be cohesionless, a nominal cohesion (c') of 2 psf is assumed in the analysis to avoid numerical instability.
The deformation characteristics E_{50}^{ref} is derived from typical values for medium dense material recommended by Kulhawy and Mayne [1990; page 5-15]. E_{oed}^{ref} is assumed to be 0.8 E_{50}^{ref} , and E_{ur}^{ref} is assumed to be 3 E_{50}^{ref} by default of PLAXIS.

Asphalt Pavement

The pavement is modeled as a flexible plate (beam) element in PLAXIS. The thickness of the asphalt pavement is assumed to be 6 in. The cross sectional area (A) and moment of inertia (I) for a unit width of the pavement (i.e., 1 ft) is calculated to be 0.5 ft^2/ft, and 0.010417 ft^4/ft. The elastic modulus of the asphalt pavement is assumed to be to be 5×10^4 psi. Accordingly, the axial stiffness (EA) and bending stiffness (EI) are calculated to be 3.6×10^6 lbs/ft and 7.5×10^4 lbs $\cdot$ ft^2/ft, respectively.

Concrete Slab

The concrete slab is also modeled as a flexible plate (beam) element in PLAXIS. The thickness of the concrete slab is assumed to be 14 in. The cross sectional area (A) and moment of inertia (I) for a unit width of the concrete slab (i.e., 1 ft) is calculated to be 1.167 ft^2/ft, and 0.132 ft^4/ft. The elastic modulus of the concrete slab (E_c) is assumed to be to be 4×10^6 psi, which is estimated from an assumed compressive strength of 5,000 psi using the following equation:

$$E_c = 33w^{1.5}\sqrt{f_c'} \quad \text{(from ACI 318-99 code)}$$

where: w = unit weight of concrete (assumed to be 145 pcf); and f_c = compressive strength

Accordingly, the axial stiffness (EA) and bending stiffness (EI) are calculated to be 6.72×10^8 lbs/ft and 7.63×10^7 lbs $\cdot$ ft^2/ft, respectively.

6 RESULTS ANALYSIS

Two different lengths of concrete slab and three different loading positions were considered in the analyses. For each cases analyzed, the vertical deflection of the pavement, horizontal displacement of the slurry wall as well as the bending moment of the concrete slab are calculated. The calculation reports generated by PLAXIS are shown in Appendix C. The report contains a summary of FEM models, input parameters, and calculation results. Note that the slurry wall corresponds to cross section A-A in the report. The results of calculation are summarized below:

Case No.	Length of Concrete Slab (ft)	Loading Position	Max. Vertical Pavement Deflection (ft)	Max. Wall Horizontal Deflection (ft)	Max Bending Moment of Slab (lbs·ft/ft)	Max Shear Force of Slab (lbs/ft)
1	10	A	0.131	0.011	**12,540**	3,140
2	10	B	**0.146**	**0.026**	1,890	2,340
3	10	C	0.137	0.024	833	739
4	20	A	**0.139**	0.013	**17,020**	3,110
5	20	B	0.134	0.015	14,720	**3,880**
6	20	C	0.136	**0.023**	4,210	2,210

As shown in the above table, the maximum horizontal deflection of the slurry cut-off wall is estimated to be 0.026 ft (0.31 in) for the case of using 10-ft long concrete slab and load applied at position B. Increasing the length of concrete slab from 10 ft to 20 ft leads to a slight reduction in the maximum horizontal deflection. However, the maximum bending moment increases by approximately 35%. The maximum vertical deflection is almost unaffected by the length of the concrete slab. Based on this calculation, 10-ft long and 20-ft long concrete slab provide almost the same degree of protection to the slurry wall. Geosyntec recommends using 10-ft long, 14-in thick reinforced concrete slab for crossing over the slurry wall.

7. REFERENCES

Kulhawy, F.H., Wayne, P.W. (1990), "*Manual on Estimating Soil Properties for Foundation Design*", prepared for Electric Power Research Institute, EPRI EL-6800.

PLAXIS BV(2006), PLAXIS version 8.2, Finite Element Code for Soil and Rock Analysis, Delft, the Netherlands.

PLAXIS BV(2006), "*Material Model Manual, PLAXIS version 8.2*", Delft, the Netherlands.

Woodward-Clyde Consultants, Inc. (1993), "*Deformation and Stability Analyses for Slurry Trench Cutoff Wall, Section 1/9 and Section 6/7, Fresh Kills Landfill*", prepared for The City of New York Department of Sanitation and International Technology Corp.

FIGURES

Written by: CL Date: 09/04/01 (YY/MM/DD) Reviewed by: RDE Date: __/__/__ (YY/MM/DD)

Client: FO Project: Fresh Kills Project/Proposal No.: ME0530B Task No: ____

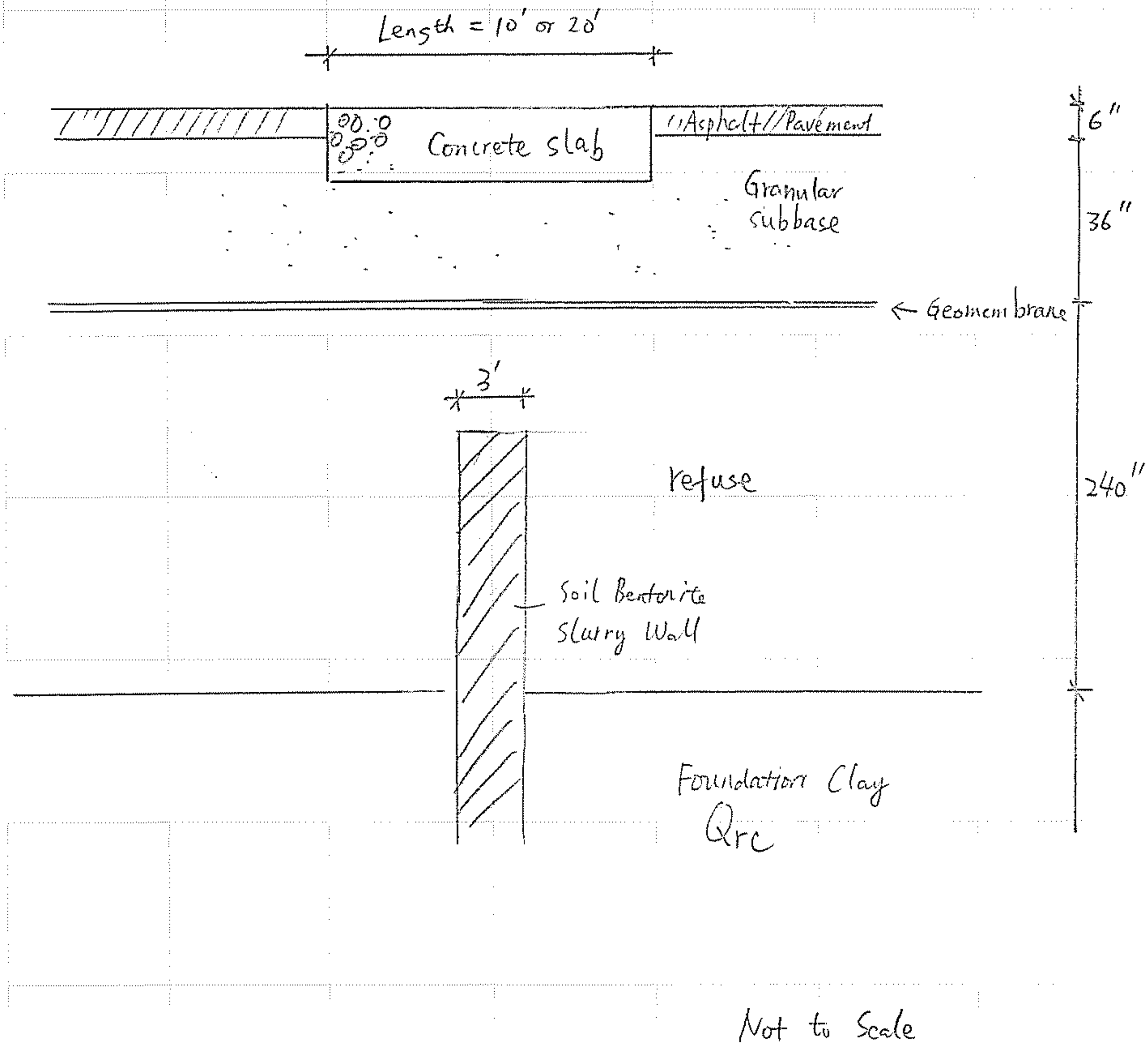

Figure 1. Idealized Cross Section for FEM Analysis

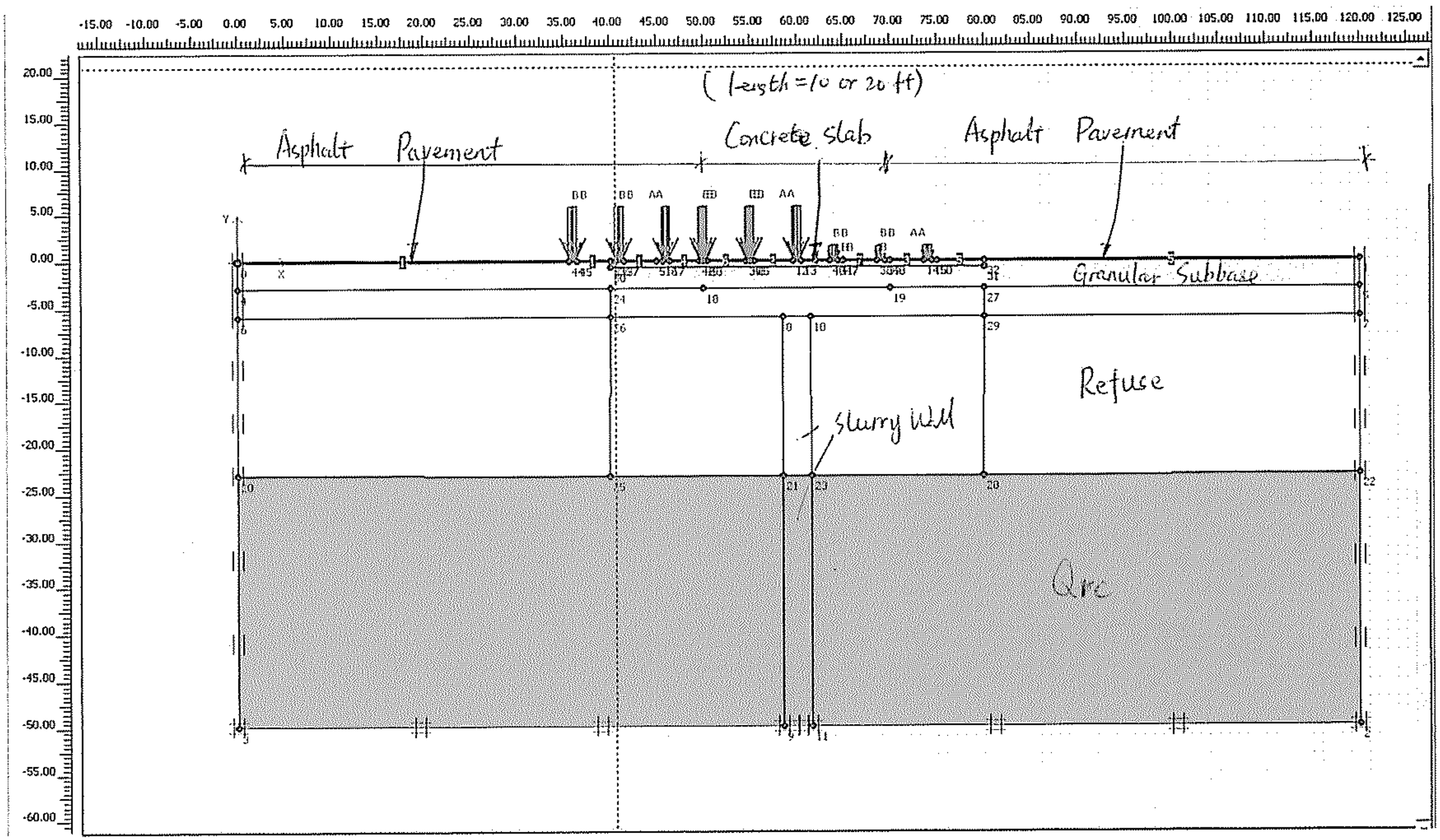

Figure 3 FEM Model

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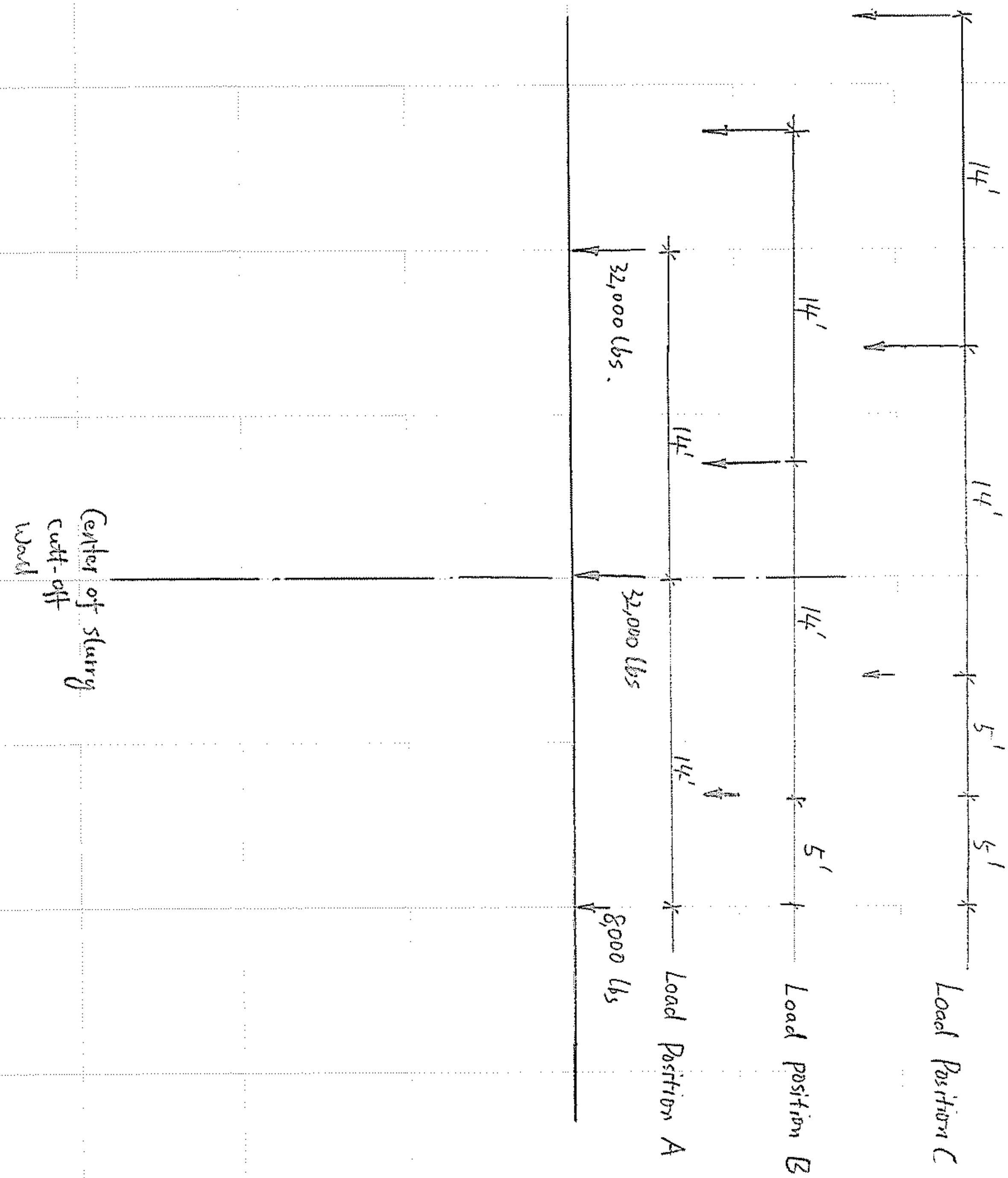


Figure 2 Illustration of Load Positions

APPENDIX A

REFERENCES ON TRAFFIC LOADING

Design Data 1

American Concrete Pipe Association

Highway Live Loads on Concrete Pipe

FOREWORD

Thick, high-strength pavements designed for heavy truck traffic substantially reduce the pressure transmitted through a wheel to the subgrade and, consequently, to the underlying concrete pipe. The pressure reduction is so great that generally the live load can be neglected. In 1926, Westergaard presented a paper summarizing the results of an extensive study of the effects of loading conditions, subgrade support, and boundary conditions on concrete pavements (1). These results formed the basis by which Westergaard developed a method to calculate the stresses in concrete slabs. Based upon the work of Westergaard and others, the Portland Cement Association, (PCA), developed a method to determine the vertical pressure on buried pipe due to wheel loads applied to concrete pavements (2). The PCA method is presented in the American Concrete Pipe Association, ACPA, "Concrete Pipe Handbook" (3) and "Concrete Pipe Design Manual" (4).

Intermediate and thin thicknesses of asphalt or flexible pavements do not reduce the pressure transmitted from a wheel to the pavement subgrade to any significant degree. For these pavements, there is no generally accepted theory for estimating load distribution effects, and, therefore, these pavements should be considered as unsurfaced roadways.

Historically, the American Association of State Highway and Transportation Officials, AASHTO, criteria for transmission of loads through soil have been published in "Standard Specifications for Highway Bridges" (5). The AASHTO Standard criteria was the primary basis for the method of determining live load pressure intensity on buried concrete pipe presented in the ACPA Handbook (3) and the ACPA Design Manual (4), with the exception that the ACPA assumes a wheel load is applied as a footprint (Figure 1), whereas the AASHTO Standard assumes a wheel load is applied as a point load. For the past decade, AASHTO has been developing a different approach to design criteria in a new publication, "Load Resistance Factor Design Bridge Design Specifications" (6), LRFD, which assumes a wheel load is applied as a footprint (Figure 1). In the future AASHTO will require all designs to be performed in accordance with the LRFD, and has stopped accepting proposed revisions to the "Standard Specifications for Highway Bridges".

This Design Data addresses the method of determining the live load pressure transmitted through unsurfaced roadways to circular, elliptical and arch concrete pipe in accordance with the criteria of the AASHTO LRFD Bridge Design Specifications .

INTRODUCTION

To determine the required supporting strength of concrete pipe installed under intermediate and thin thicknesses of asphalt or flexible pavements, or relatively shallow earth cover, it is necessary to evaluate the effect of live loads, such as highway truck loads, in addition to dead loads imposed by the soil and surcharge loads.

LIVE LOADS

If a rigid pavement or a thick flexible pavement designed for heavy duty traffic is provided with a sufficient buffer between the pipe and pavement, then the live load transmitted through the pavement to the buried concrete pipe is usually negligible at any depth. If any culvert or sewer pipe is within the heavy duty traffic highway right-of-way, but not under the pavement structure, then such pipe should be analyzed for the effect of live load transmission from an unsurfaced roadway, because of the possibility of trucks leaving the pavement.

DEAD LOADS

Various methods for analyzing soil dead loads, which have been developed over the years, are presented in the ACPA "Concrete Pipe Technology Handbook" (7).

SURCHARGE LOADS

A common type of surcharge load is additional soil fill placed after the pipe has been installed for a period of time. If the surcharge load is a building or other surface load, the resultant uniformly distributed load can be converted to an equivalent height of fill, and then evaluated as an additional soil load. When concrete pipe has been installed underground, the soil-structure system will continually show an increase in load capacity. Data on concrete pipe, which have been removed from service and tested, indicate an increase in concrete strength and an increase in load carrying capacity of 10 to 40 percent. Settlement and consolidation will improve the soil structure surrounding the pipe, which also improves load carrying capacity.

LIVE LOADS

The AASHTO design loads commonly used in the past were the HS 20 with a 32,000 pound axle load in the Normal Truck Configuration, and a 24,000 pound axle load in the Alternate Load Configuration (Figure 2).

The AASHTO LRFD design loads are the HS 20 with a 32,000 pound axle load in the Normal Truck Configuration, and a 25,000 pound axle load in the Alternate Load Configuration (Figure 2). In addition, the AASHTO LRFD requires the application of a 640 pound per linear foot Lane Load applied across a 10 foot wide lane at all depths of earth cover over the top of the pipe, up to a depth of 8 feet. This Lane Load converts to an additional live load of 64 pounds per square foot, applied to the top of the pipe for any depth of burial less than 8 feet. The average pressure intensity caused by a wheel load is calculated by Equation 2. The Lane Load intensity is added to the wheel load pressure intensity in Equation 3.

The HS 20, 32,000 pound and the Alternate Truck 25,000 pound design axle are carried on dual wheels (Figure 1). The contact area of the dual wheels with the ground is assumed to be a rectangle (Figure 1), with dimensions presented in Table 1.

Figure 1 AASHTO Wheel Load Surface Contact Area (Foot Print)

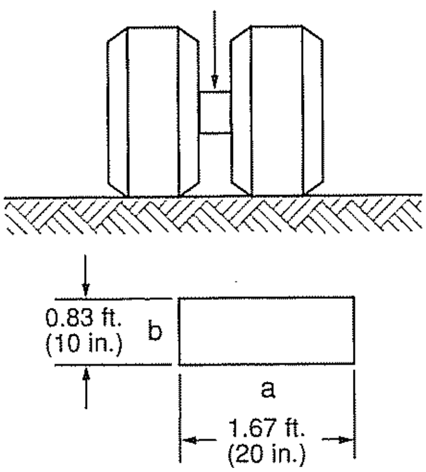


Table 1 LRFD Wheel Surface Contact Area

a (width), ft (in.)	b (length), ft (in.)
1.67(20)	0.83(10)

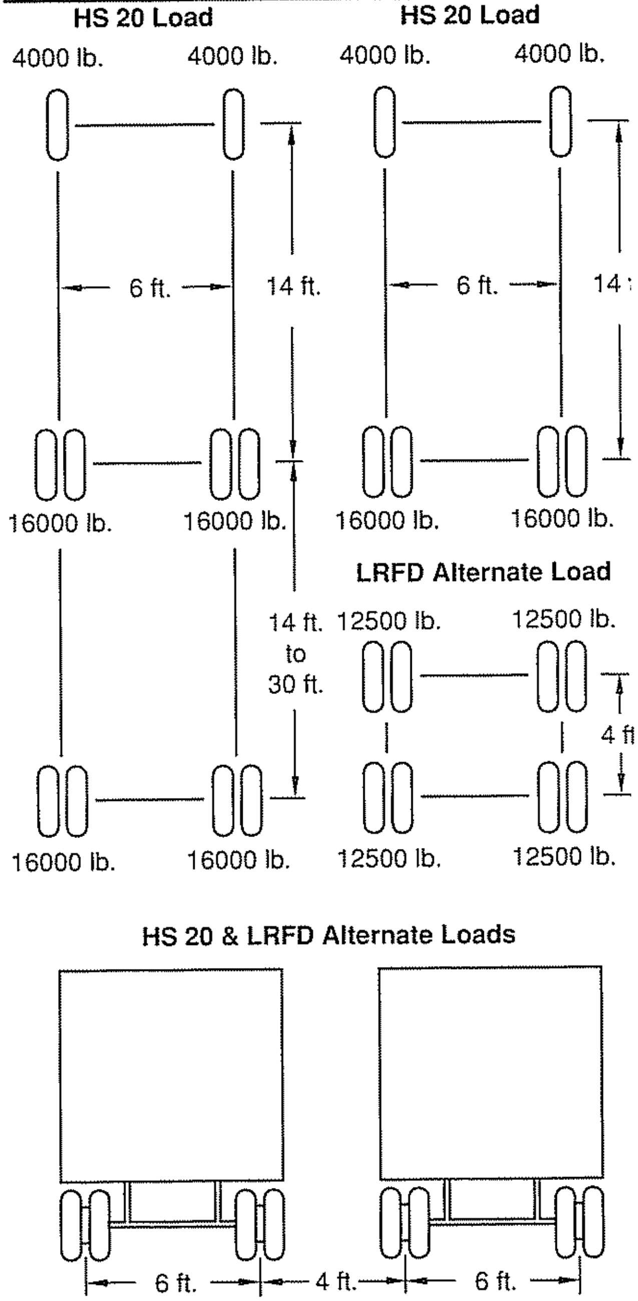
IMPACT FACTORS

The AASHTO LRFD Standard applies a dynamic load allowance to account for the truck load being non-static. The dynamic load allowance, IM, is determined by Equation 1:

$$IM = \frac{33(1.0 - 0.125H)}{100} \quad [1]$$

where: H = height of earth cover over the top of the pipe, ft.

Figure 2 AASHTO Wheel Loads and Wheel Spacings



LOAD DISTRIBUTION

The surface load is assumed to be uniformly spread on any horizontal subsoil plane. The spread load area is developed by increasing the length and width of the wheel contact area for a load configuration as illustrated in Figure 3 for a dual wheel; in Figure 4 for dual wheels of two trucks in passing mode; and in Figure 5 for two dual wheels of two Alternate Load configurations in passing mode. On a horizontal soil plane, the dimensional increases to the wheel contact area are based on height of earth cover over the top of the pipe as presented in Table 2 for two types of soil.

Table 2 LRFD Wheel Contact Area Dimensional Increase Factor

Soil Type	Dimensional Increase Factor
LRFD select granular	1.15H
LRFD any other soil	1.00H

As indicated by Figures 3, 4 and 5, the spread load areas from adjacent wheels will overlap as the height of earth cover over the top of the pipe increases. At shallow depths, the maximum pressure will be developed by an HS 20 dual wheel, since at 16,000 pounds it applies a greater load than the 12,500 pound Alternate Load (Figures 2 and 3).

Table 3 LRFD Critical Wheel Loads and Spread Dimensions at the Top of the Pipe for Select Granular Soil Fill

H, ft	P, lbs	Spread a, ft	Spread b, ft	Figure
$H < 2.03$	16,000	a + 1.15H	b + 1.15H	3
$2.03 \leq H < 2.76$	32,000	a + 4 + 1.15H	b + 1.15H	4
$2.76 \leq H$	50,000	a + 4 + 1.15H	b + 4 + 1.15H	5

Table 4 LRFD Critical Wheel Loads and Spread Dimensions at the Top of the Pipe for Other Soils

H, ft	P, lbs	Spread a, ft	Spread b, ft	Figure
$H < 2.33$	16,000	a + 1.00H	b + 1.00H	3
$2.33 \leq H < 3.17$	32,000	a + 4 + 1.00H	b + 1.00H	4
$3.17 \leq H$	50,000	a + 4 + 1.00H	b + 4 + 1.00H	5

Figure 3 Spread Load Area - Single Dual Wheel

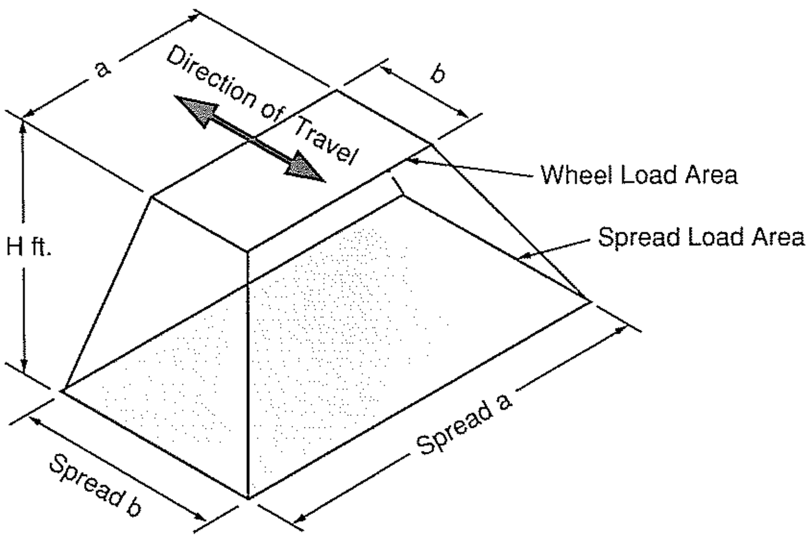


Figure 4 Spread Load Area - Two Single Dual Wheels of Trucks in Passing Mode

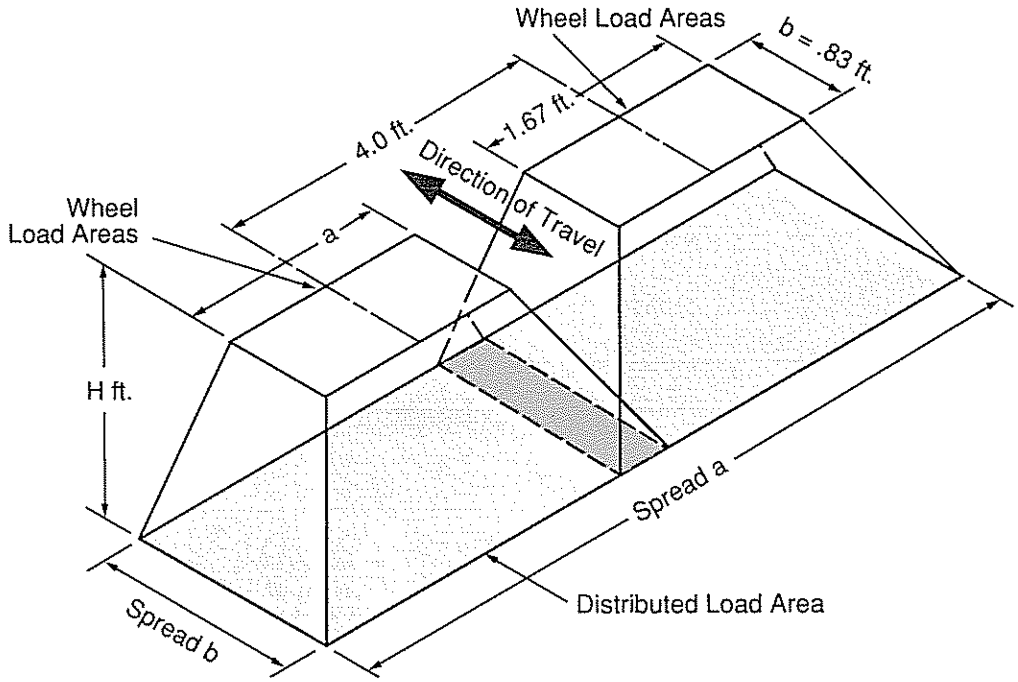
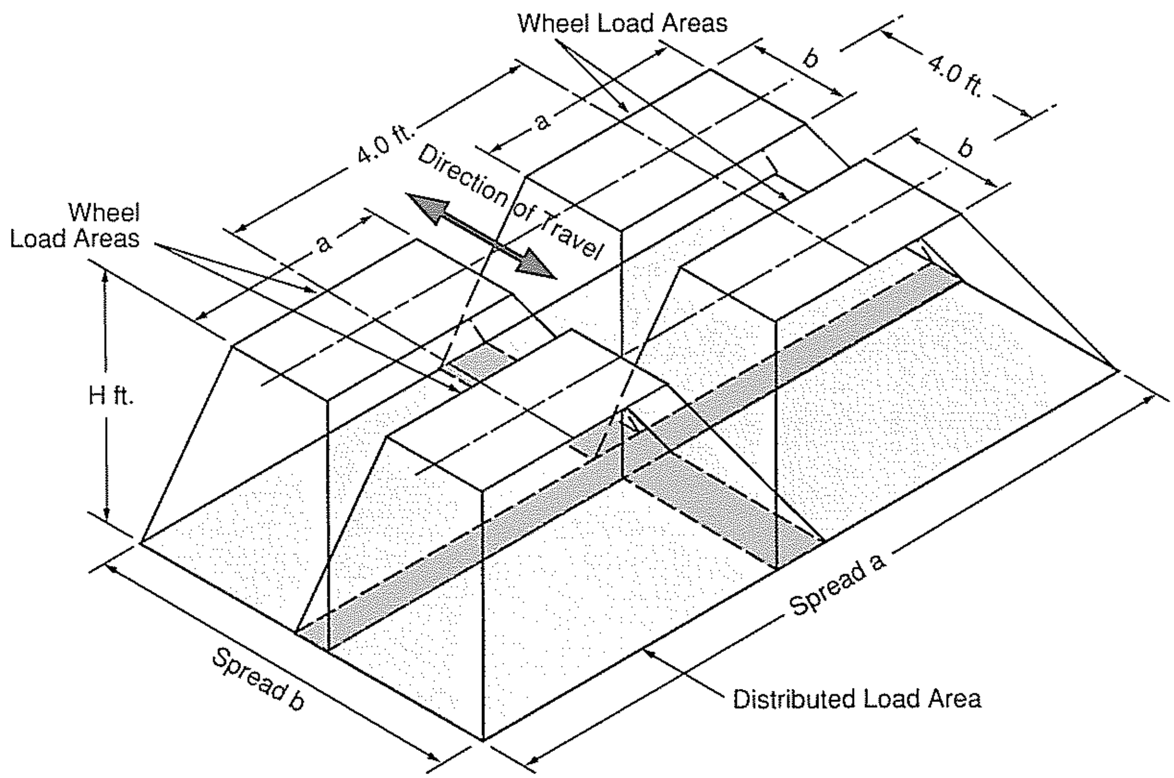


Figure 5 Spread Load Area - Two Single Dual Wheels of Two Alternate Loads in Passing Mode



At intermediate depths, the maximum pressure will be developed by the wheels of two HS 20 trucks in the passing mode, since at 16,000 pounds each, the two wheels apply a greater load than the 12,500 pounds of an Alternate Load wheel (Figures 2 and 4). At greater depths, the maximum pressure will be developed by wheels of two Alternate Load configuration trucks in the passing mode, since at 12,500 pounds each, the four wheels apply the greatest load (50,000 pounds) (Figures 2 and 5). Intermediate depths begin when the spread area of dual wheels of two HS 20 trucks in the passing mode meet and begin to overlap. Greater depths begin when the spread area b of two single dual wheels of two Alternate Load configurations in the passing mode meet and begin to overlap.

Since the exact geometric relationship of individual or combinations of surface wheel loads cannot be anticipated, the most critical loading configurations along with axle loads and rectangular spread load area are presented in Tables 3 and 4 for the two AASHTO LRFD soil types.

DESIGN METHOD

The design method encompasses 4 steps.

1. Obtain the following project data:
   Pipe shape, size and wall thickness.
   Height of cover over the concrete pipe, and type of earth fill.
   LRFD or other criteria.
2. Calculate the average pressure intensity of the wheel loads on the soil plane on the outside top of the pipe.
3. Calculate the total live wheel load and lane load acting on the pipe.
4. Calculate the total live load acting on the pipe in pounds per linear foot.

Project Data

Pipe shape and internal dimensions are shown on the project plans. Complete information on dimensional details are included in ASTM Specification C 14 for nonreinforced circular concrete pipe (8), C 76 for reinforced concrete circular pipe (9), C 506 for reinforced concrete arch pipe (10) and C 507 for reinforced concrete elliptical pipe (11). Internal size, wall thickness and outside dimensions are presented in Tables 6, 7 and 8 for circular, arch and elliptical pipe respectively.

The minimum earth cover over the concrete pipe can be obtained from the project plans. The type of fill material required under, around and over the concrete pipe will be noted on the project plans or detailed in the contract documents.

A decision regarding whether the AASHTO LRFD or other criteria will be used should be obtained from the project authority.

Average Pressure Intensity

The wheel load average pressure intensity on the subsoil plane at the outside top of the concrete pipe is:

$$w = \frac{P(1 + IM)}{A} \qquad [2]$$

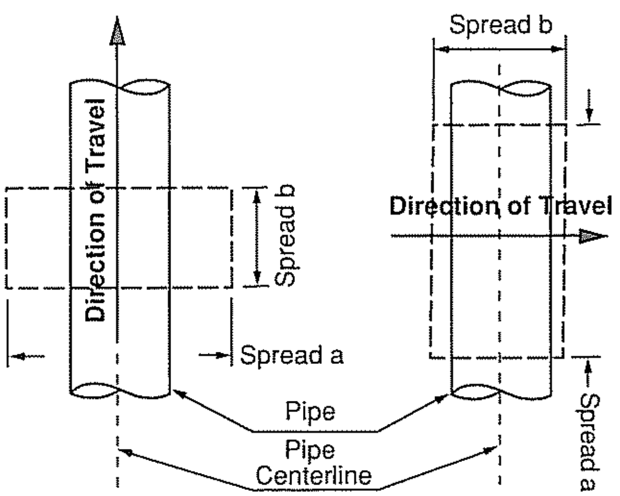
where:
- w = wheel load average pressure intensity, pounds per square foot
- P = total live wheel load applied at the surface, pounds
- A = spread wheel load area at the outside top of the pipe, square feet
- IM = dynamic load allowance

From the appropriate Table 3, or 4, select the critical wheel load and spread dimensions for the height of earth cover over the outside top of the pipe, H. The spread live load area is equal to Spread a times Spread b. Select the appropriate dynamic load allowance, using Equation 1.

Total Live Load

A designer is concerned with the maximum possible loads, which occur when the distributed load area is centered over the buried pipe. Depending on the pipe size and height of cover, the most critical loading orientation can occur either when the truck travels transverse or parallel to the centerline of the pipe. Figure 6 illustrates the dimensions of the spread load area, A, as related to whether the truck travel is transverse or parallel to the centerline of the pipe.

Figure 6 Spread Load Area Dimensions vs Direction of Truck



Unless you are certain of the pipeline orientation, the total live load in pounds, W_T, must be calculated for each travel orientation, and the maximum calculated value must be used in Equation 4 to calculate the live load on the pipe in pounds per linear foot.

The LRFD requires a Lane Load, L_L, of 64 pounds per square foot on the top of the pipe at any depth less than 8 feet.

The total live load acting on the pipe is :

$$L_e = L + 1.75(3/4R_0) \quad [5]$$

where:
- W_T =total live load, pounds
- w =wheel load average pressure intensity, pounds per square foot (at the top of the pipe)
- L_L =lane loading if AASHTO LRFD is used, pounds per square foot
- $0 \leq H < 8$, L_L=64, pounds per square foot
- $H \geq 8$, L_L =0
- L =dimension of A parallel to the longitudinal axis of pipe, feet
- S_L =outside horizontal span of pipe, B_c, or dimension of A transverse to the longitudinal axis of pipe, whichever is less, feet

Total Live Load in Pounds per Linear Foot

The total live load in pounds per linear foot, W_L, is calculated by dividing the Total Live Load, W_T, by the Effective Supporting Length, L_e (See Figure 7), of the pipe:

$$W_L = \frac{W_T}{L_e} \quad [4]$$

where:
- W_L =live load on top of pipe, pounds per linear foot
- L_e =effective supporting length of pipe (see Figure 7), feet

The effective supporting length of pipe is:

$$L_e = L + 1.75(3/4R_0) \quad [5]$$

where: R_o =outside vertical Rise of pipe, feet

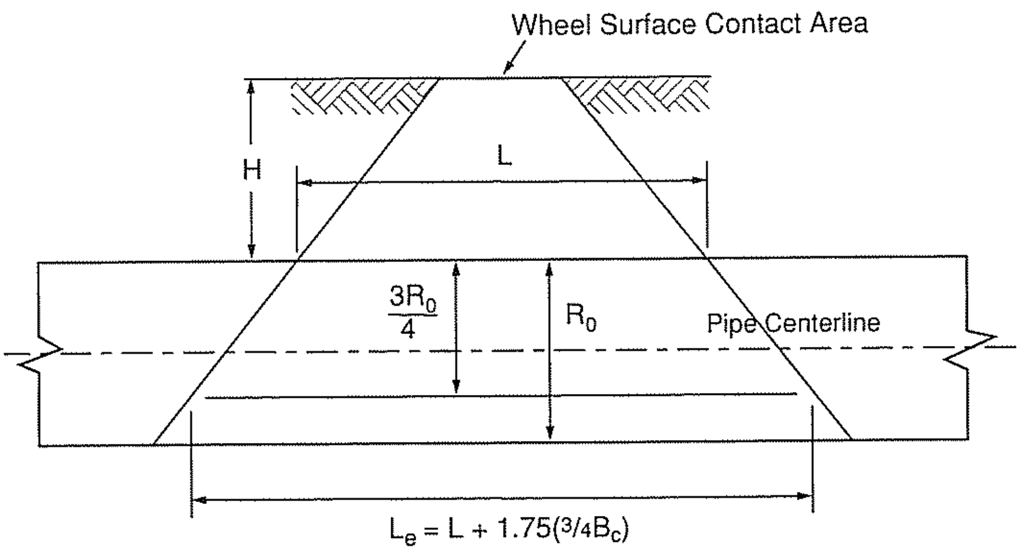
EXAMPLES

Four Example calculations are presented on the following pages to illustrate the four steps of the Design Method, and the effect of varying the depth of fill and the type of fill. The live loads per linear foot calculated in the four Examples are summarized in Table 5.

Table 5 Summary of LRFD Live Loads Calculated in Examples

Soil Fill	D,in	H, ft	P, lbs	Live Load, plf
Select Granular	30	2	16,000	2559
Other	30	2	16,000	2672
Select Granular	30	4	50,000	1471
Other	30	4	50,000	1622

Figure 7 Effective Supporting Length of Pipe



EXAMPLE 1

Given: A 30-inch diameter, B wall, concrete pipe is to be installed as a storm drain under a flexible pavement and subjected to AASHTO highway loadings. The pipe will be installed in a trench with a minimum of 2 feet of cover over the top of the pipe. The AASHTO LRFD Criteria will be used with Select Granular Soil.

Find: The maximum live load on the pipe in pounds per linear foot.

Solution:

1. Review project data.

A 30-inch diameter, B wall, circular concrete pipe has a wall thickness of 3.5 inches, therefore B_c is 3.08 feet and R_o is 3.08 feet. Height of earth cover is 2 feet. Use AASHTO LRFD Criteria with Select Granular Soil Fill.

2. Calculate average pressure intensity of the live load on the plane at the outside top of the pipe.

From Table 3, the critical load, P, is 16,000 pounds from an HS 20 single dual wheel, and the Spread Area is:

A=(Spread a)(Spread b)
A=(1.67 + 1.15x2)(0.83 + 1.15x2)
A=(3.97)(3.13)
A=12.4 square feet

I.M.=33(1.0 - 0.125H)/100
I.M.=.2475 (24.75%)
w=P(1 + IM)/A
w=16000(1 + .2475)/12.4
w=1,610 lb/ft^2

3. Calculate total live load acting on the pipe.

$W_T = (w + L_L)LS_L$

Assuming truck travel transverse to pipe centerline.

L_L=64
L=Spread a = 3.97 feet
Spread b=3.13 feet
B_c=3.08 feet, which is less than Spread b, therefore
S_L=3.08 feet

W_T = (1610 + 64)x3.97x3.08 = 20,500 pounds

Assuming truck travel parallel to pipe centerline.

L_L=64
Spread a=3.97 feet
L=Spread b = 3.13 feet
B_c=3.08 feet, which is less than Spread a, therefore
S_L=3.08 feet

W_T = (1610 + 64) 3.08 x 3.13 = 16,100 pounds

W_T Maximum = 20,500 pounds; and truck travel is transverse to pipe centerline

4. Calculate live load on pipe in pounds per linear foot.

R_o=3.08 feet

L_e=L + 1.75(3/4R_o)
L_e=3.97 + 1.75(.75x3.08) = 8.01 feet

W_L=W_T/L_e
W_L=20,500/8.01 = 2,559 pounds per linear foot

EXAMPLE 2

Given: Same as Example 1, except use AASHTO LRFD Criteria with Other Soils Fill.

Find: The maximum live load on the pipe in pounds per linear foot.

Solution:

1. Review project data.

A wall B 30-inch diameter circular concrete pipe has a wall thickness of 3.5 inches, therefore B_c is 3.08 feet and R_o is 3.08 feet. Height of earth cover is 2 feet. Use AASHTO LRFD Criteria with Other Soils Fill.

2. Calculate average pressure intensity on the plane at the top of the pipe.

From Table 4, the critical load, P, is 16,000 pounds from an HS 20 single dual wheel, and the Spread Area is:

A=(Spread a)(Spread b)
A=(1.67 + 1.00x2)(0.83 + 1.00x2)
A=(3.67)(2.83)
A=10.4 square feet

I.M.=33(1.0 - 0.125H)/100
I.M.=.2475

w=P(I + IM)/A
w=16,000(1 + .2475)/10.4
w=1,920 lb/ft^2

3. Calculate total live load acting on the pipe.

$W_T = (w + L_L)LS_L$

Assuming truck travel transverse to pipe centerline.
L_L=64
L=Spread a = 3.67 feet
Spread b=2.83 feet
B_c=3.08 feet, which is greater than Spread b, therefore
S_L=2.83 feet

W_T=(1,920 + 64)x3.67x2.83 = 20,600 pounds

Assuming truck travel parallel to pipe centerline.

L_L=64
Spread a=3.67 feet
L=Spread b = 2.83 feet
B_c=3.08 feet, which is less than Spread a, therefore
S_L=3.08 feet

W_T = (1,920 + 64)x2.83x3.08 = 17,300 pounds

W_T Maximum = 20,600 pounds; and truck travel is transverse to pipe centerline

4. Calculate live load on pipe in pounds per linear foot.

R_o=3.08 feet

L_e=L + 1.75(3/4R_o)
L_e=3.67 + 1.75(.75x3.08) = 7.71 feet

$W_L=W_T/L_e$
W_L=20,600/7.71 = 2,672 pounds per linear foot

EXAMPLE 3

Given: Same as Example 1, except minimum depth of fill is 4 feet.

Find: The maximum live load on the pipe in pounds per linear foot.

Solution:

1. Review project data.

A wall B 30-inch diameter circular concrete pipe has a wall thickness of 3.5 inches, therefore B_c is 3.08 feet and R_o is 3.08 feet. Height of earth cover is 4 feet. Use AASHTO LRFD Criteria with Select Granular Soil Fill.

2. Calculate average pressure intensity at the outside top of the pipe.

From Table 3, the critical load, P, is 50,000 pounds from two single dual wheels of two Alternate Load Configurations in the passing mode, and the Spread Area is:

A=(Spread a)(Spread b)
A=(1.67 + 4 + 1.15x4)(0.83 + 4 + 1.15x4)
A=(10.27)(9.43)
A=96.85 square feet

I.M.=33(1-0.125H)/100
I.M.=0.165

w=P(1 + I.M.)/A = 50,000(1 +.165)/96.85
w=601 lb/ft^2

3. Calculate total live load acting on the pipe.

$W_T = (w + L_L)LS_L$

Assuming truck travel transverse to pipe centerline.

L_L=64
L=Spread a = 10.27 feet
Spread b=9.43 feet
B_c=3.08 feet, which is less than Spread b, therefore
S_L=3.08 feet

W_T = (601 + 64)x10.27x3.08 = 21,035 pounds

Assuming truck travel parallel to pipe centerline.

L_L=64
Spread a=10.27 feet
L=Spread b = 9.43 feet
B_c=3.08 feet, which is less than Spread a, therefore
S_L=3.08 feet

W_T = (601 + 64)x9.43x3.08 = 19,315 pounds

W_T Maximum=21,035 pounds; and truck travel is transverse to pipe centerline

4. Calculate live load on pipe in pounds per linear foot.

R_o=3.08 feet

L_e=L + 1.75(3/4R_o)
L_e=10.27 + 1.75(0.75x3.08) = 14.3 feet

$W_L = W_T / L_e$
$W_L = 21{,}035/14.3 = 1{,}471$ pounds per linear foot

EXAMPLE 4

Given: Same as Example 2, except minimum depth of fill is 4 feet.

Find: The maximum live load on the pipe in pounds per linear foot.

Solution:

1. Review project data.

A wall B 30-inch diameter circular concrete pipe has a wall thickness of 3.5 inches, therefore B_c is 3.08 feet and R_o is 3.08 feet. Height of earth cover is 8 feet. Use AASHTO LRFD Criteria with Other Soils Fill.

2. Calculate average pressure intensity on the plane at the top of the pipe.

From Table 4, the critical load, P, is 50,000 pounds from two single dual wheels of two Alternate Load Configurations in the passing mode, and the Spread Area is:

A = (Spread a)(Spread b)
A = (1.67 + 4 + 1.00x4)(0.83 + 4 + 1.00x4)
A = (9.67)(8.83)
A = 85.4 square feet

I.M. = 0.165

w = P(l + IM)/A
w = 50,000(1+ .165)/85.4
w = 682 lb/ft^2

3. Calculate total live load acting on the pipe.

$W_T = (w + L_L)LS_L$

Assuming truck travel transverse to pipe centerline.

L_L=64
L=Spread a = 9.67 feet
Spread b=8.83 feet
B_c=3.08 feet, which is less than Spread b, therefore
S_L=3.08 feet

W_T = (682 + 64)x9.67x3.08 = 22,219 pounds

Assuming truck travel parallel to pipe centerline.

L_L=64
Spread a=9.67 feet
L=Spread b = 8.83 feet
B_c=3.08 feet, which is less than Spread a, therefore
S_L=3.08 feet

W_T = (682 + 64)x8.83x3.08 = 20,289 pounds

W_T Maximum = 22,219 pounds; and truck travel is transverse to pipe centerline.

4. Calculate live load on pipe in pounds per linear foot.

R_o=3.08 feet

$L_e = L + 1.75(3/4R_o$
$L_e = 9.67 + 1.75(0.75x3.08) = 13.7$ feet

$W_L = W_T / L_e$
$W_L = 22{,}219/13.7 = 1{,}622$ pounds per linear foot

Table 6 Dimensions of Circular Concrete Pipe

Internal Diameter, inches	Wall A Minimum Wall Thickness, inches	Wall B Minimum Wall Thickness, inches	Wall C Minimum Wall Thickness, inches
12	1-3/4	2	-
15	1-7/8	2-1/4	-
18	2	2-1/2	-
21	2-1/4	2-3/4	-
24	2-1/2	3	3-3/4
27	2-5/8	3-1/4	4
30	2-3/4	3-1/2	4-1/4
33	2-7/8	3-3/4	4-1/2
36	3	4	4-3/4
42	3-1/2	4-1/2	5-1/4
48	4	5	5-3/4
54	4-1/2	5-1/2	6-1/4
60	5	6	6-3/4
66	5-1/2	6-1/2	7-1/4
72	6	7	7-3/4
78	6-1/2	7-1/2	8-1/4
84	7	8	8-3/4
90	7-1/2	8-1/2	9-1/4
96	8	9	9-3/4
102	8-1/2	9-1/2	10-1/4
108	9	10	10-3/4
114	9-1/2	10-1/2	11-1/4
120	10	11	11-3/4
126	10-1/2	11-1/2	12-1/4
132	11	12	12-3/4
138	11-1/2	12-1/2	13-1/4
144	12	13	13-3/4
150	12-1/2	13-1/2	14-1/4
156	13	14	14-3/4
162	13-1/2	14-1/2	15-1/4
168	14	15	15-3/4
174	14-1/2	15-1/2	16-1/4
180	15	16	16-3/4

Table 7 Dimensions of Arch Concrete Pipe

Equivalent Round Size, inches	Minimum Rise, inches	Minimum Span, inches	Minimum Wall Thickness, inches
15	11	18	2-1/4
18	13-1/2	22	2-1/2
21	15-1/2	26	2-3/4
24	18	28-1/2	3
30	22-1/2	36-1/4	3-1/2
36	26-5/8	43-3/4	4
42	3-15/16	5-1/8	4-1/2
48	36	58-1/2	5
54	40	65	5-1/2
60	45	73	6
72	54	88	7
84	62	102	8
90	72	115	8-1/2
96	77-1/4	122	9
108	87-1/8	138	10
120	96-7/8	154	11
132	106-1/2	168-3/4	10

Table 7 Dimensions of Elliptical Concrete Pipe

Equivalent Round Size, inches	Minor Axis, inches	Major Axis inches	Minimum Wall Thickness, inches
18	14	23	2-3/4
24	19	30	3-1/4
27	22	34	3-1/2
30	24	38	3-1/4
33	27	42	3-3/4
36	29	45	4-1/2
39	32	49	4-3/4
42	34	53	5
48	38	60	5-1/2
54	43	68	6
60	48	76	6-1/2
66	53	83	7
72	58	91	7-1/2
78	63	98	8
84	68	106	8-1/2
90	72	113	9
96	77	121	9-1/2
102	82	128	9-3/4
108	87	136	10
114	92	143	10-1/2
120	97	151	11
132	106	166	12
144	116	180	13

References

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2. "Vertical Pressure on Culverts Under Wheel Loads on Concrete Pavement Slabs", Portland Cement Association, 1944.

3. *"Concrete Pipe Handbook"*, American Concrete Pipe Association, 1998.

4. *"Concrete Pipe Design Manual"*, American Concrete Pipe Association, 2000.

5. *"Standard Specifications for Highway Bridges"*, American Association for State Highway and Transportation Officials.

6. *"LRFD Bridge Design Specifications"*, American Association for State Highway and Transportation Officials.

7. *"Concrete Pipe Technology Handbook"*, American Concrete Pipe Association, 1993.

8. ASTM Standard C 14, "Specification for Concrete Sewer, Storm Drain, and Culvert Pipe", American Society for Testing and Materials.

9. ASTM Standard C 76, "Specification for Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe", American Society for Testing and Materials.

10. ASTM Standard C 506, "Specification for Reinforced Concrete Arch Culvert, Storm Drain, and Sewer Pipe", American Society for Testing and Materials.

11. ASTM Standard C 507, "Specification for Reinforced Concrete Elliptical Culvert, Storm Drain, and Sewer Pipe", American Society for Testing and Materials.

APPENDIX B

INPUT PARAMETERS FOR FEM MODEL

TABLE 3-1

SUMMARY OF THE FINITE ELEMENT HYPERBOLIC PARAMETERS
CUTOFF WALL DEFORMATION AND STABILITY ANALYSES
FRESH KILLS LANDFILL

Parameter	Name	Relationships
K_m	modulus number	$E_i = K_m P_a (\sigma_3/P_a)^n$ $E_{ur} = K_{ur} P_a (\sigma_3/P_a)^n$
K_{ur}	unload/reload modulus number	
n	modulus exponent	
c	cohesion intercept	failure stress, $(\sigma_1 - \sigma_3)_f = \dfrac{2c\cos\phi + 2\sigma_3 \sin\phi}{1 - \sin\phi}$
ϕ	friction angle	
R_f	failure ratio	$R_f = \dfrac{\text{failure stress, } (\sigma_1 - \sigma_3)_f}{\text{ultimate stress, } (\sigma_1 - \sigma_3)_{ult}}$
K_b	bulk modulus number	$B = K_b P_a (\sigma_3 / P_a)^m$
m	bulk modulus exponent	

Notes:

E_i = initial modulus
E_{ur} = unload/reload modulus
P_a = atmospheric pressure
B = bulk modulus
σ_3 = confining pressure

TABLE 4-1

SUMMARY OF FINITE ELEMENT MODEL PARAMETERS
CUTOFF WALL DEFORMATION AND STABILITY ANALYSES
FRESH KILLS LANDFILL

	Recent Refuse	Decomposed Refuse	Qrc	Qgl	Qrs	Qgt/Kc/TrSc	Soil-Bentonite Backfill	Cement-Bentonite Backfill
Unit Weight, γ (pcf)	73	73	94	126.5	125	135	105	125
Cohesion Intercept, c (psf)	300	0	0	150	0	0	20	5,800
Friction Angle, ϕ (deg.)	28	28	36	24	35	40	0	0
Coefficient of Earth Pressure at Rest, K_o	0.5	0.5	0.7	0.8	0.45	1.2	1.0	1.0
Modulus Exponent, n	0.45	0.45	0.85	0.65	0.25	0.6	0	0
Modulus Number, K_m	35	35	17	70	450	1,000	2	N/A
Unload/Reload Modulus Number, K_{ur}	55	55	26	105	650	1,500	3	N/A
Failure Ratio, R_f	0.7	0.7	0.7	0.7	0.7	0.5	0.7	N/A
Minimum Initial Modulus, $E_{i,min}$ (psf)	25,000	25,000	20,000	80,000	500,000	1,000,000	4,000	500,000
Bulk Modulus Exponent, m	0.3	0.3	0.2	0.2	0.0	0.2	N/A	N/A
Bulk Modulus Number, K_b	15	15	15	60	350	8,000	N/A	N/A
Modulus at Failure, E_f (psf)	250	250	200	800	5,000	1,000	40	500,000

Power m for hardening soil model → (Modulus Exponent, n)

Input Parameters for Hardening Soil Model

$E_{50}^{ref} = K_m \cdot P_a$

$E_{ur}^{ref} = K_{ur} \cdot P_a$

$E_{oed}^{ref} = 0.8\, E_{50}^{ref}$

	refuse	Qrc	Soil-bentonite back fill
E_{50}^{ref}	= 35×2116 = 70 ksf	= 17 X 2116 = 34 ksf	= 2×2116 = 4.2 ksf.
E_{ur}^{ref}	= 55×2116 = 116 ksf (used 157 ksf based on Plaxis recommendation)	= 26 X 2116 ≐ 52 ksf (Used 68 ksf based on Plaxis recommendation)	
E_{oed}^{ref}	= 70 ksf × 0.8 = 56 ksf	= 0.8×34 = 27 ksf	

TABLE 4-2
SLOPE STABILITY PARAMETERS
FRESH KILLS LANDFILL

Material	Saturated Unit Weight γ (pcf)	Consolidation Parameters					ESA Strength Parameters		USA Strength Parameters	Comments
		Ave. Maximum Past Pressure (ksf)	CR	RR	Cv x 10^-4 (cm²/sec)	Cvs	ϕ' (degrees)	c' (ksf)	Su (ksf)	
Refuse Fill										See Note 1
Recent	73	n/a	n/a	n/a	n/a	n/a	33	0.3	n/a	
Decomposed	73	n/a	0.49	0.07	40-3000	n/a	28	0	n/a	
Qrc	94	1.5	0.27	0.05	3	n/a	36	0	0.28 σ'	
Qrn/Qgs	125	n/a	n/a	n/a	n/a	n/a	35	n/a	n/a	Assumed Incompressible
Qgl										
Low Plastic (w/o CH soils)	126.5	23	0.12	0.02	181	164	25	0	0.21 σ' +0.65	
Plastic (w/ CH soils)	126.5	23	0.14	0.03	90	63	23	0.15	0.10 σ' +1.0	
Soil -Bentonite Backfill	105	n/a	n/a	n/a	n/a	n/a	0.0 *	0.0 *	0.0 *	

Notes: 1) Decomposed Refuse Fill considered below 1/3 of the landfill height.
Recent Refuse Fill considered above 2/3 of the landfill height.
2) CR - Virgin Compression Ratio
3) RR - Recompression Ratio
4) Cv - Normally Consolidated Coefficient of Consolidation
5) Cvs - Overconsolidated Coefficient of Consolidation
6) Su - Undrained Shear Strength
* - See text for explanation of strength parameters.

Table 5-5

TYPICAL RANGES OF DRAINED MODULUS FOR SAND

Consistency	Normalized Elastic Modulus, E_d/p_a	
	Typical	Driven Piles[a]
loose	100 to 200	275 to 550
medium	200 to 500	550 to 700
dense	500 to 1000	700 to 1100

a - Source: Poulos (17), p. 207.

Used $E_d = 350 \cdot 2116 \approx 700$ ksf (p_a)

$$:p_a\ (\bar{\sigma}_3/p_a)^n\ [1 - R_f\ (1 - \sin \bar{\phi}_{tc})(\bar{\sigma}_1 - \bar{\sigma}_3)/(2\ \bar{\sigma}_3 \sin \bar{\phi}_{tc})]^2 \qquad (5\text{-}21)$$

. and $\bar{\sigma}_3$ = effective major and minor principal stresses, respectively, :tive stress friction angle in triaxial compression, and κ, n, and R_f = 'ameters given in Table 5-6. For convenience in computer code implemen- .utmann and Kulhawy (1) approximated κ as follows:

$$0 + 900\ \phi_{rel} \qquad (5\text{-}22)$$

efined in Equation 5-8.

s with Strength

odulus commonly is correlated to the effective soil strength through the dex (I_r), as defined below for drained loading:

$$/(\bar{\sigma}\ \tan \bar{\phi}_{tc}) \qquad (5\text{-}23)$$

lues for I_r are given in Table 5-7. Of particular interest to note is reases with increasing relative density and decreases with increasing ss. It also is lower with more compressible soil minerals.

:he rigidity index (I_r) for drained loading, volume changes normally :onsidered. Therefore, I_r must be corrected for the volumetric strains ld a reduced rigidity index (I_{rr}), as given below by Vesić (20):

APPENDIX C

PLAXIS REPORTS

REPORT

05/20/2009

User: GeoSyntec Consultants

Title: 10-ft long concrete slab

Table of Contents

1. General Information

Table [1] Model

Model	Plane Strain
Element	15-Noded

2. Geometry

2.1. Clusters

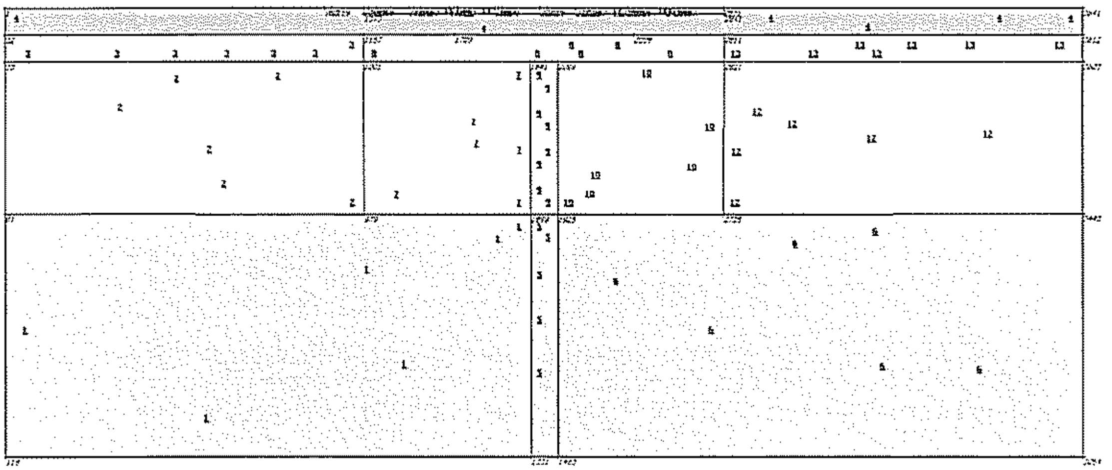

Fig. 1 Plot of geometry model with cluster numbers

3. Material data

Table [2] Soil data sets parameters

Mohr-Coulomb		1 **Soil-Bentonite Backfill**
Type		Drained
γ_{unsat}	[lb/ft³]	105.00
γ_{sat}	[lb/ft³]	105.00
k_x	[ft/day]	0.000
k_y	[ft/day]	0.000
e_{init}	[-]	0.500
c_k	[-]	1E15
E_{ref}	[lb/ft²]	4200.000
ν	[-]	0.200
G_{ref}	[lb/ft²]	1750.000
E_{oed}	[lb/ft²]	4666.667
c_{ref}	[lb/ft²]	20.00
φ	[°]	0.50
ψ	[°]	0.00
E_{inc}	[lb/ft²/ft]	0.00
y_{ref}	[ft]	0.000
$c_{increment}$	[lb/ft²/ft]	0.00
$T_{str.}$	[lb/ft²]	0.00
$R_{inter.}$	[-]	1.00
Interface permeability		Neutral

Hardening Soil		2 **granular fill**	3 **refuse**	4 **Qrc**
Type		Drained	Drained	Drained
γ_{unsat}	[lb/ft³]	120.00	73.00	94.00
γ_{sat}	[lb/ft³]	120.00	73.00	94.00
k_x	[ft/day]	0.000	0.000	0.000
k_y	[ft/day]	0.000	0.000	0.000
e_{init}	[-]	0.50	0.50	0.50
e_{min}	[-]	0.00	0.00	0.00
e_{max}	[-]	999.00	999.00	999.00
c_k	[-]	1E15	1E15	1E15
E_{50}^{ref}	[lb/ft²]	700000.00	70000.00	34000.00
E_{oed}^{ref}	[lb/ft²]	560000.00	56000.00	27000.00
power (m)	[-]	0.50	0.45	0.85
c_{ref}	[lb/ft²]	2.00	5.00	5.00
φ	[°]	34.00	28.00	36.00
ψ	[°]	0.00	0.00	0.00
E_{ur}^{ref}	[lb/ft²]	2100000.00	157000.00	68000.00
$\nu_{ur}^{(nu)}$	[-]	0.200	0.200	0.200
p^{ref}	[lb/ft²]	2116.00	2116.00	100.00
$c_{increment}$	[lb/ft²]	0.00	0.00	0.00
y_{ref}	[ft]	0.00	0.00	0.00
R_f	[-]	0.90	0.90	0.90
$T_{str.}$	[lb/ft²]	0.00	0.00	0.00
R_{inter}	[-]	1.00	1.00	1.00

Hardening Soil	2 **granular fill**	3 **refuse**	4 **Qrc**
δ_{inter} [ft]	0.00	0.00	0.00
Interface permeability	Neutral	Neutral	Neutral

Table [3] Beam data sets parameters

no.	Identification	EA [lb/ft]	EI [lbft²/ft]	w [lb/ft/ft]	ν [-]	Mp [lbft/ft]	Np [lb/ft]
1	Concrete Plate	6.72E8	7.3E7	175.00	0.20	1E15	1E15
2	pavement	3.6E6	75000.00	60.00	0.20	1E15	1E15

4. Calculation phases

Table [4] List of phases

Phase	Ph-No.	Start phase	Calculation type	Load input	First step	Last step
Initial phase	0	0		-	0	0
Slurry Wall installation	1	0	Plastic analysis	Staged construction	1	114
Surcharge	2	1	Plastic analysis	Staged construction	115	130
surcharge removed	3	2	Plastic analysis	Staged construction	131	166
Load Position A	4	3	Plastic analysis	Staged construction	167	186
Load Position B	5	3	Plastic analysis	Staged construction	187	215
Load Position C	6	3	Plastic analysis	Staged construction	216	236

5. Results for phase 4

5.2. Deformations

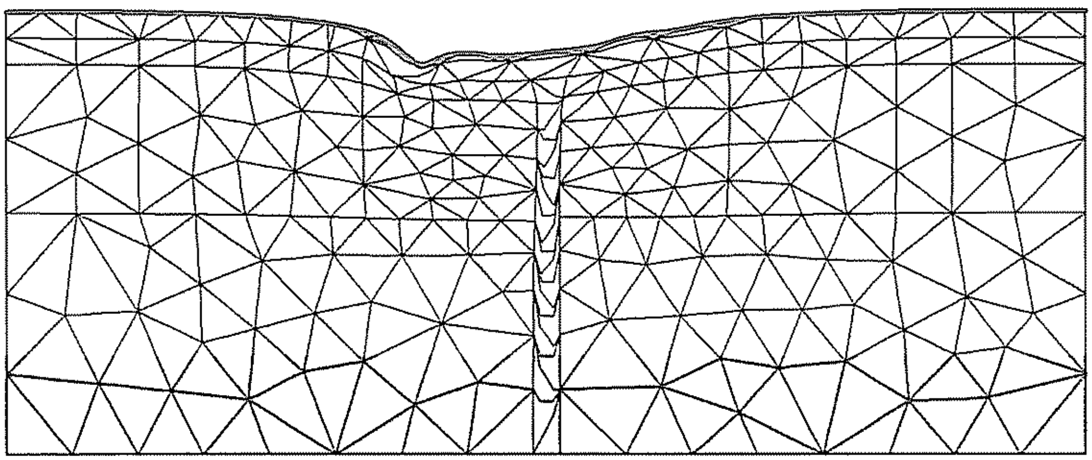

Fig. 2 Plot of deformed mesh
- Step no: 186 - (Phase: 4)

5.2.1. Plot of total displacements

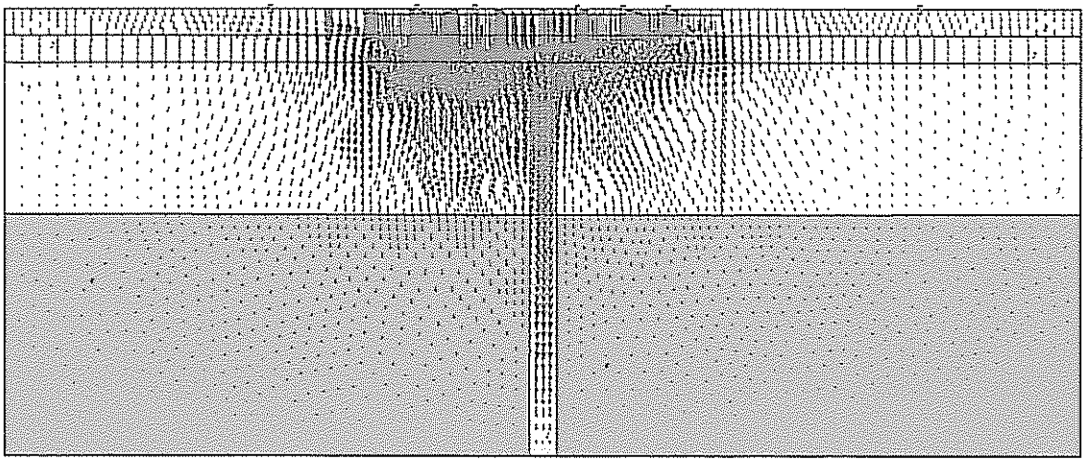

Fig. 3 Plot of total displacements (arrows)
- Step no: 186 - (Phase: 4)

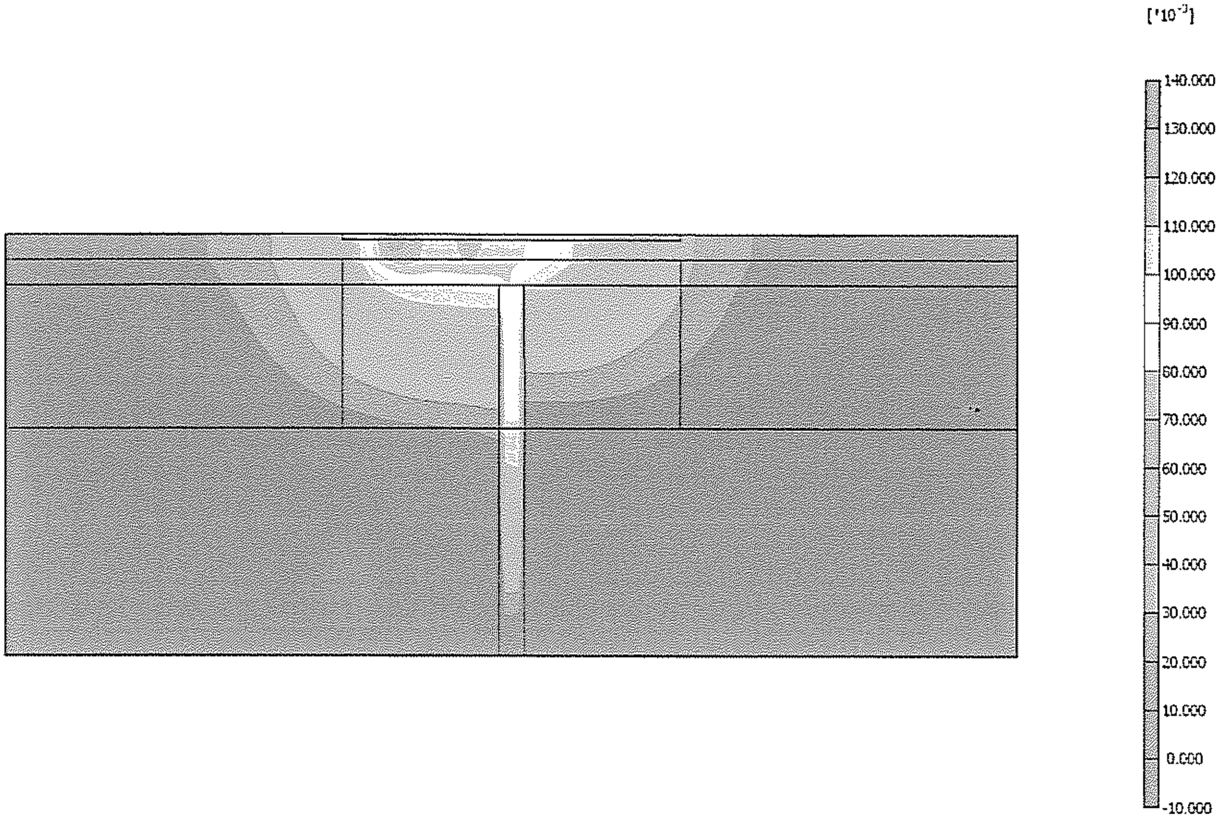

Fig. 4 Plot of total displacements (shadings)
- Step no: 186 - (Phase: 4)

5.2.2. Plot of vertical displacements

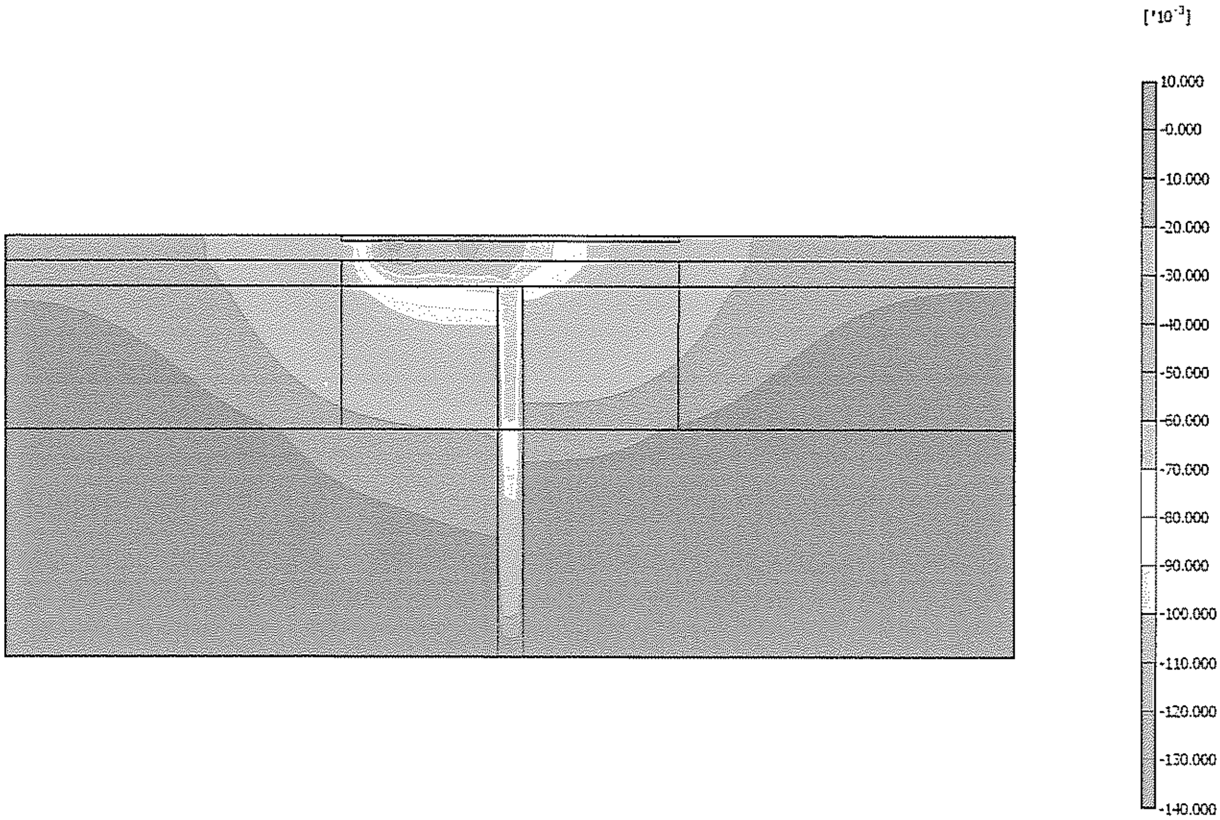

Fig. 5 Plot of vertical displacements (shadings)
- Step no: 186 - (Phase: 4)

5.3. Structures

5.3.3. Beams

5.3.3.1. Beams

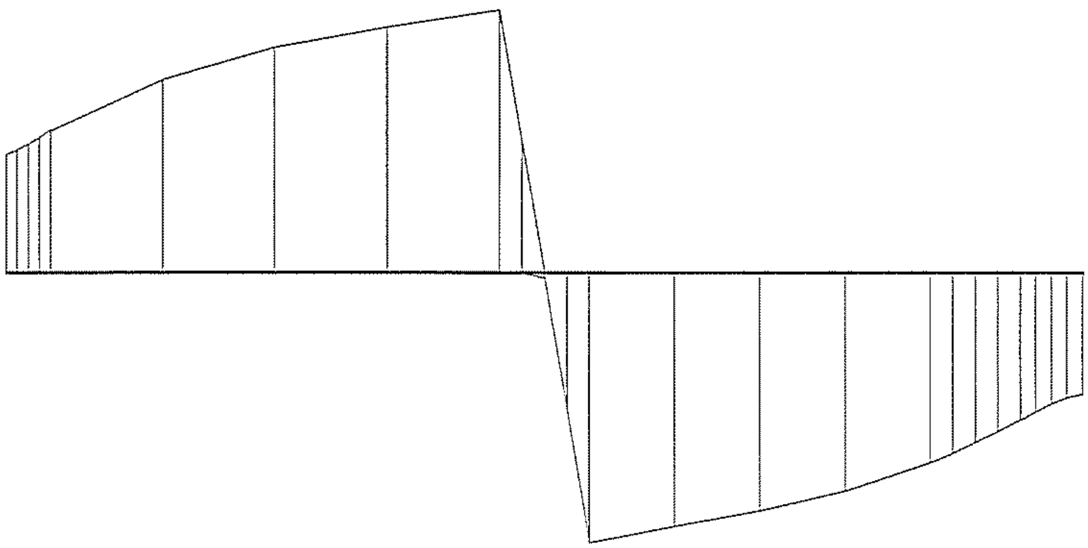

Fig. 6 Shear forces envelop in beam (plate no: 2)
Extreme value $3.14*10^3$ lb/ft (Phase: 4)

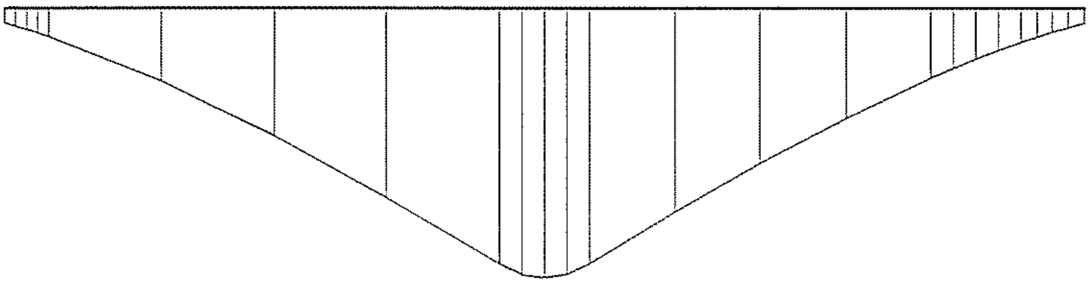

Fig. 7 Bending moment envelop in beam (plate no: 2)
Extreme value $12.54*10^3$ lb/ft/ft (Phase: 4)

5.4. Cross sections

5.4.4. Deformations

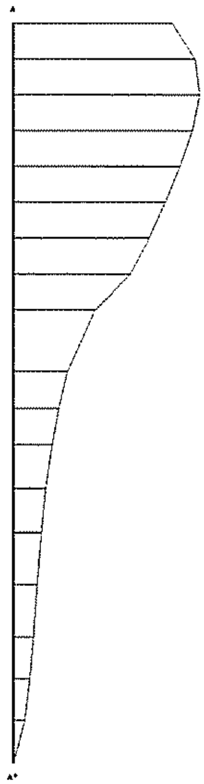

Fig. 8 Horizontal displacements in cross section (Cross Section A - A*)
Extreme value $11.47*10^{-3}$ ft (Phase: 4)

6. Results for phase 5

6.5. Deformations

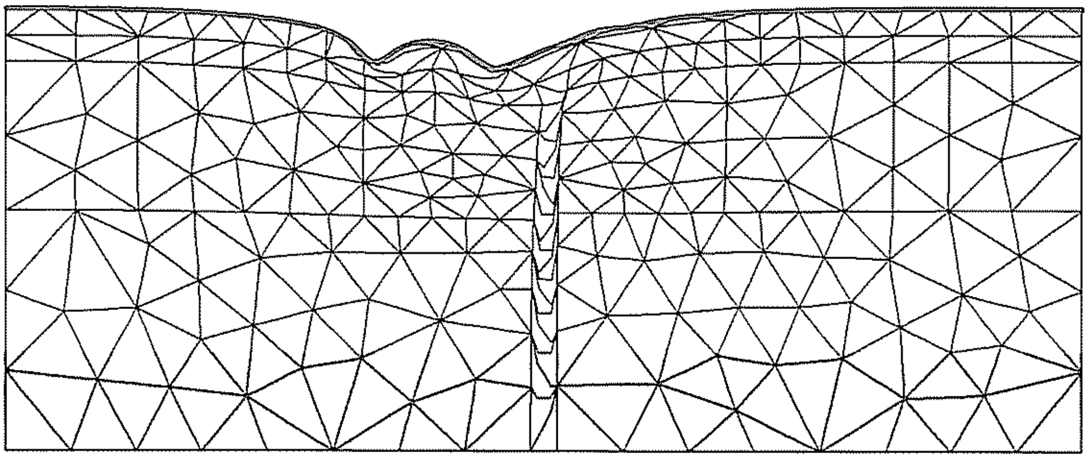

Fig. 9 Plot of deformed mesh
- Step no: 215 - (Phase: 5)

6.5.5. Plot of total displacements

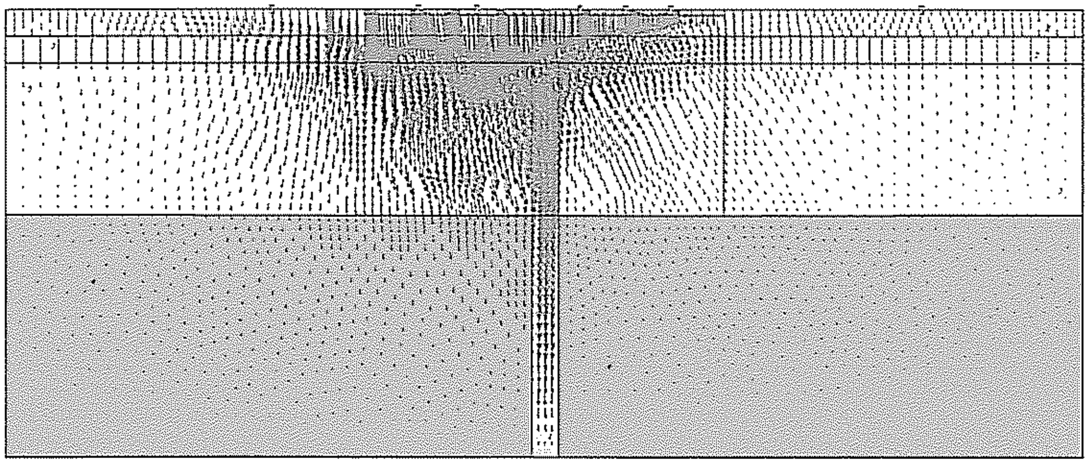

Fig. 10 Plot of total displacements (arrows)
- Step no: 215 - (Phase: 5)

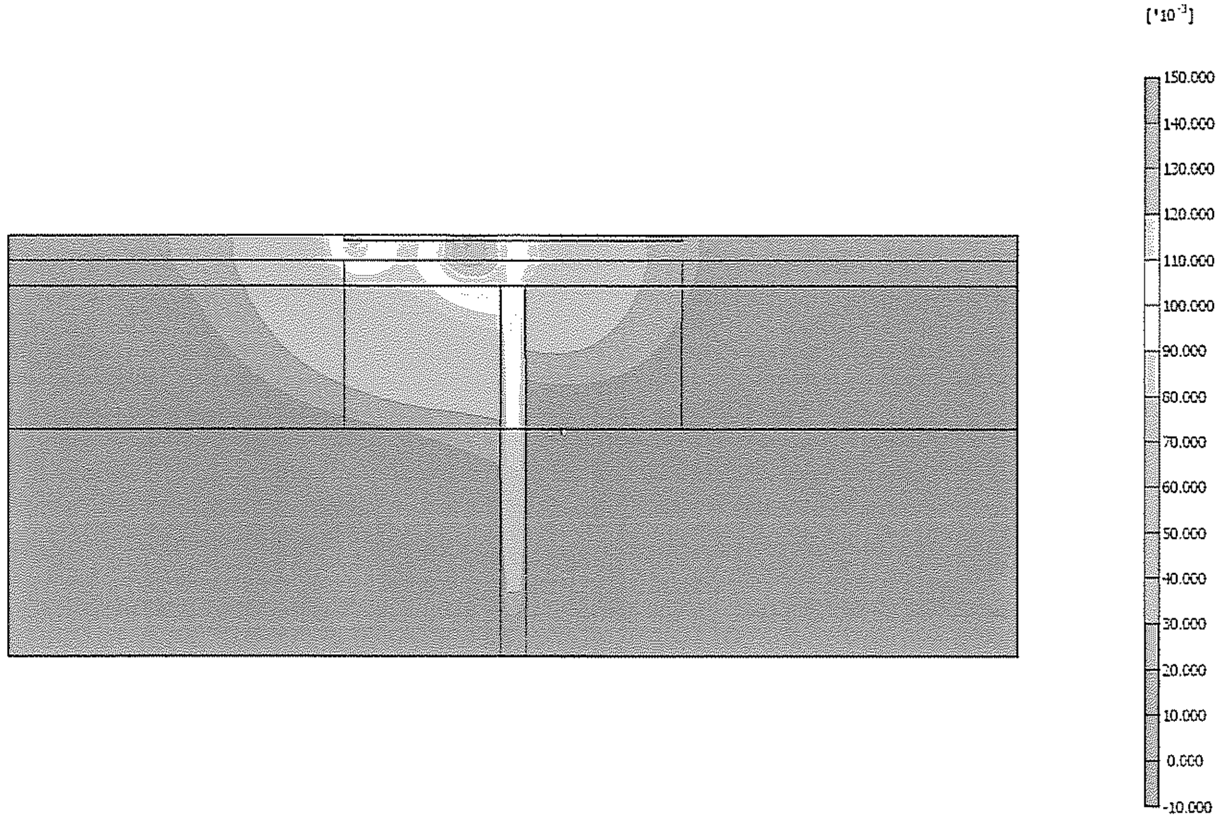

Fig. 11 Plot of total displacements (shadings)
- Step no: 215 - (Phase: 5)

6.5.6. Plot of vertical displacements

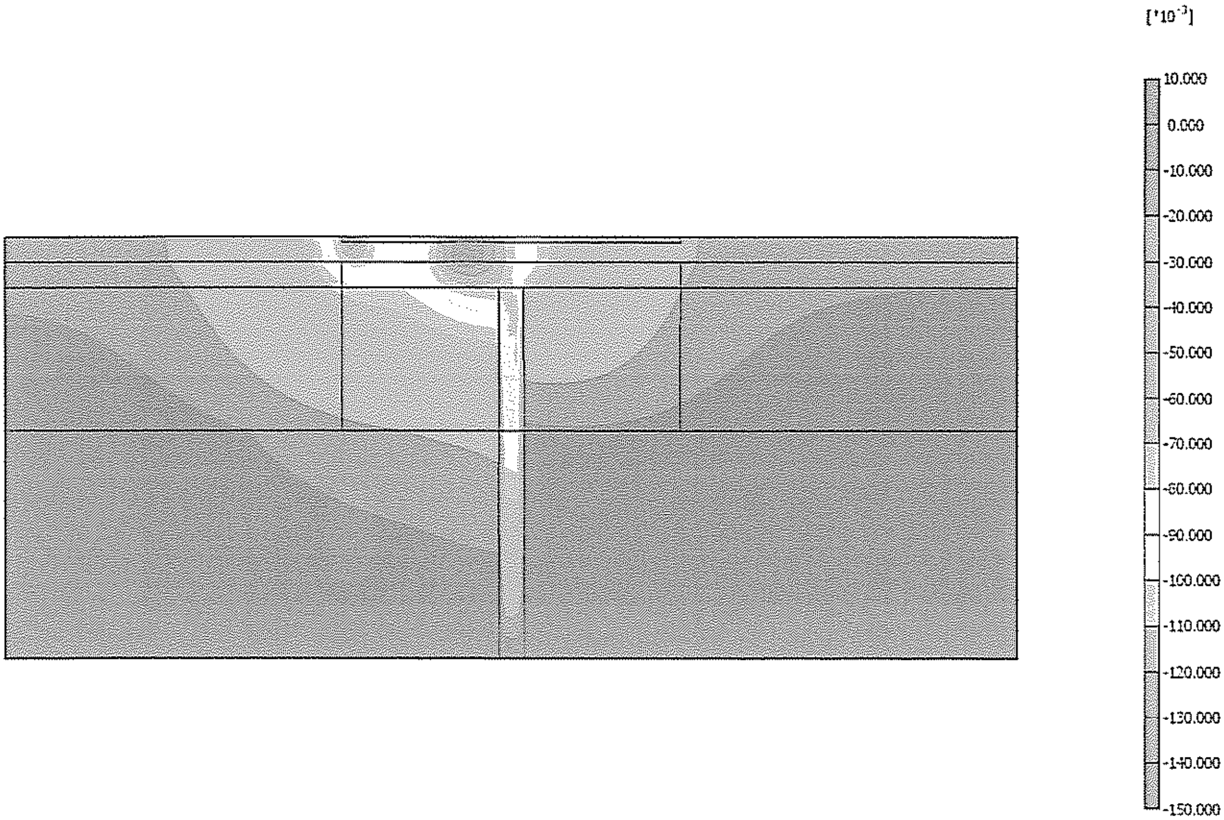

Fig. 12 Plot of vertical displacements (shadings)
- Step no: 215 - (Phase: 5)

6.6. Structures

6.6.7. Beams

6.6.7.2. Beams

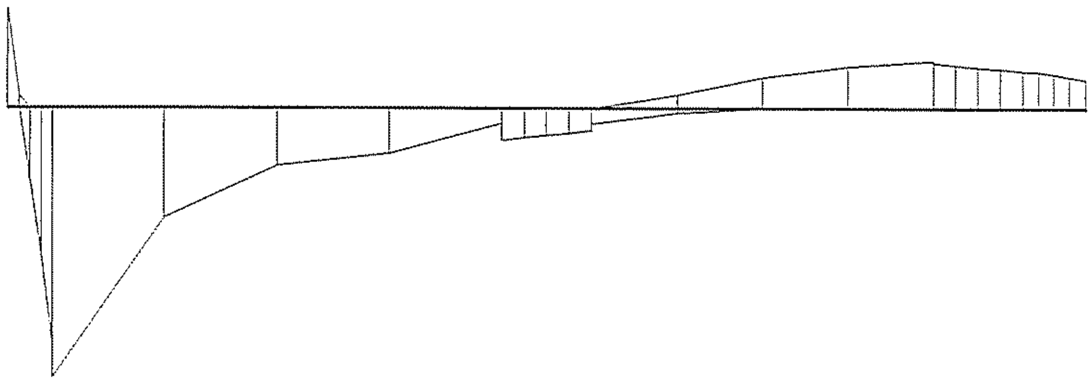

Fig. 13 Shear forces envelop in beam (plate no: 2)
Extreme value $2.34*10^3$ lb/ft (Phase: 5)

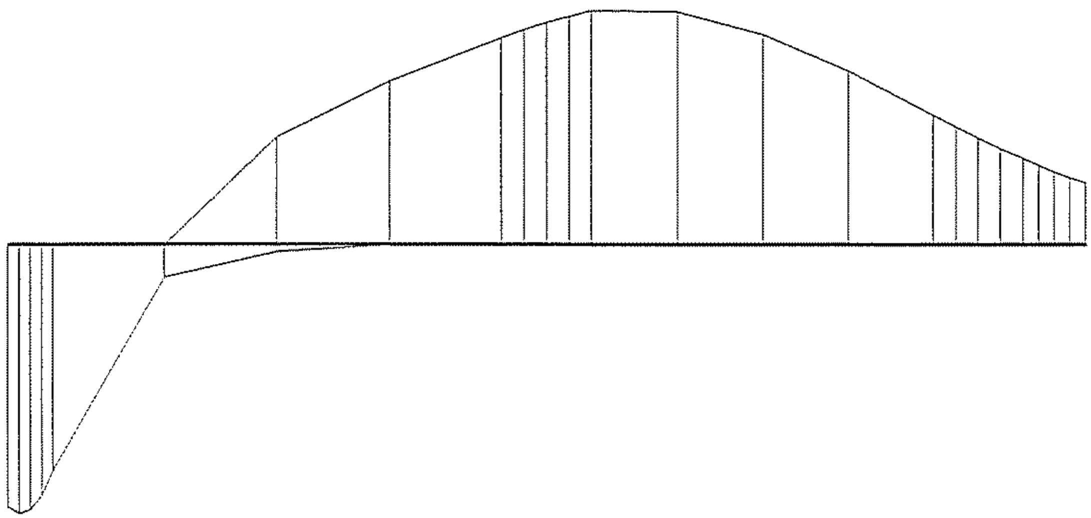

Fig. 14 Bending moment envelop in beam (plate no: 2)
Extreme value 1.89*10^3 lb/ft/ft (Phase: 5)

6.7. Cross sections

6.7.8. Deformations

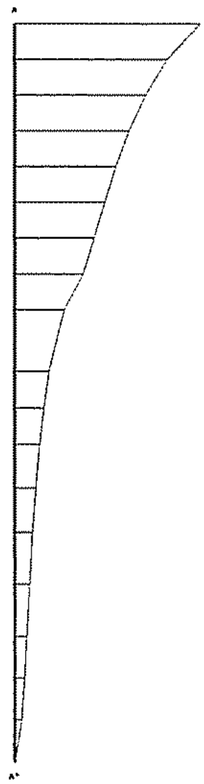

Fig. 15 Horizontal displacements in cross section (Cross Section A - A*)
Extreme value 26.22*10^{-3} ft (Phase: 5)

7. Results for phase 6

7.8. Deformations

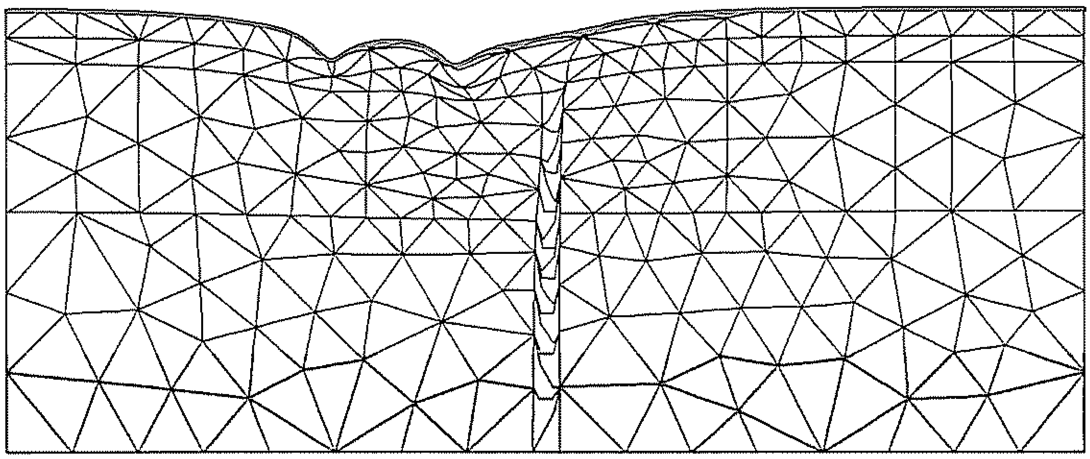

Fig. 16 Plot of deformed mesh
- Step no: 236 - (Phase: 6)

7.8.9. Plot of total displacements

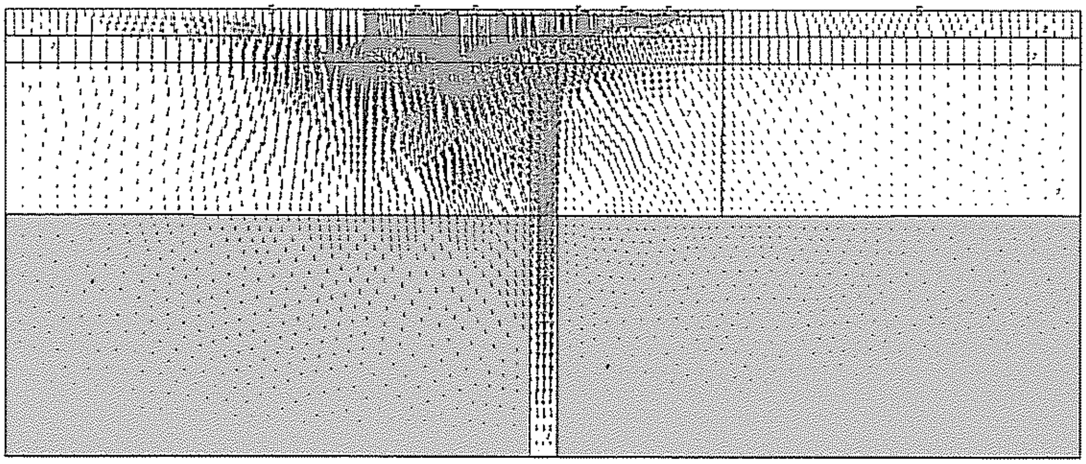

Fig. 17 Plot of total displacements (arrows)
- Step no: 236 - (Phase: 6)

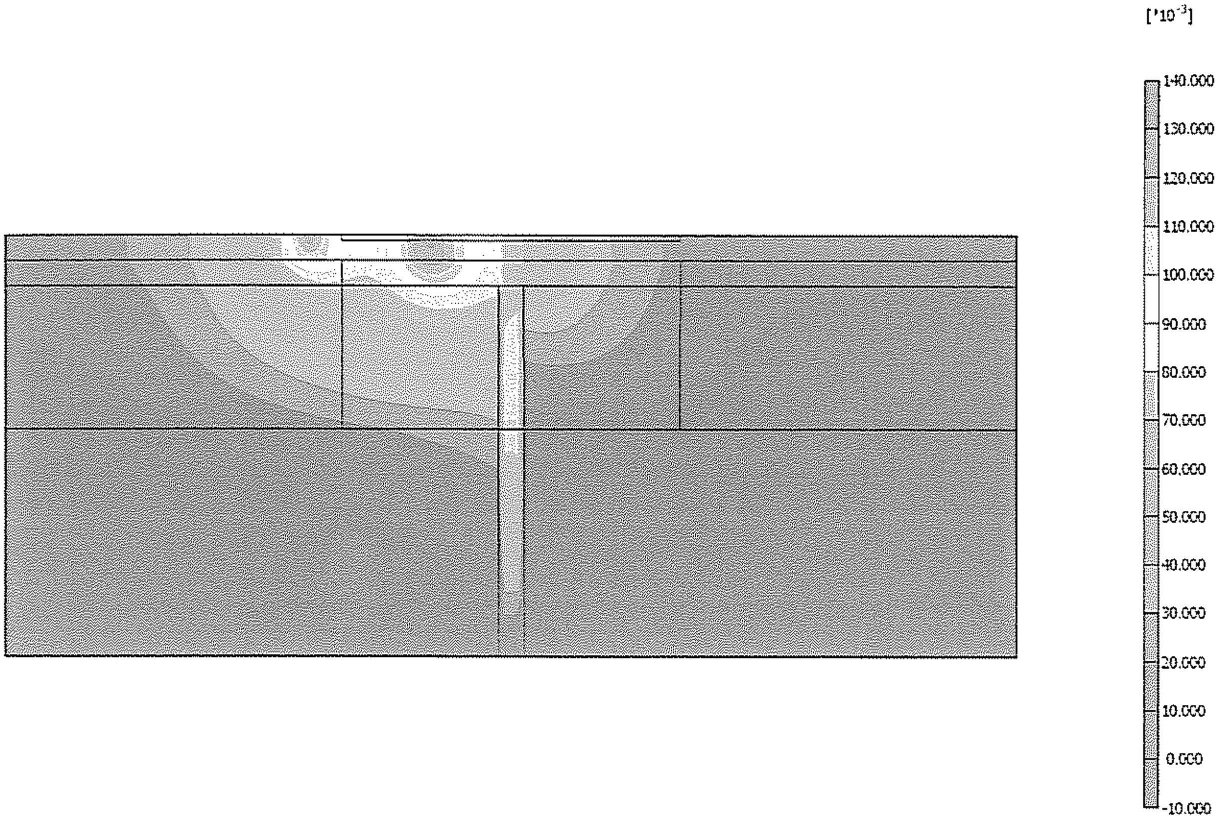

Fig. 18 Plot of total displacements (shadings)
- Step no: 236 - (Phase: 6)

7.8.10. Plot of vertical displacements

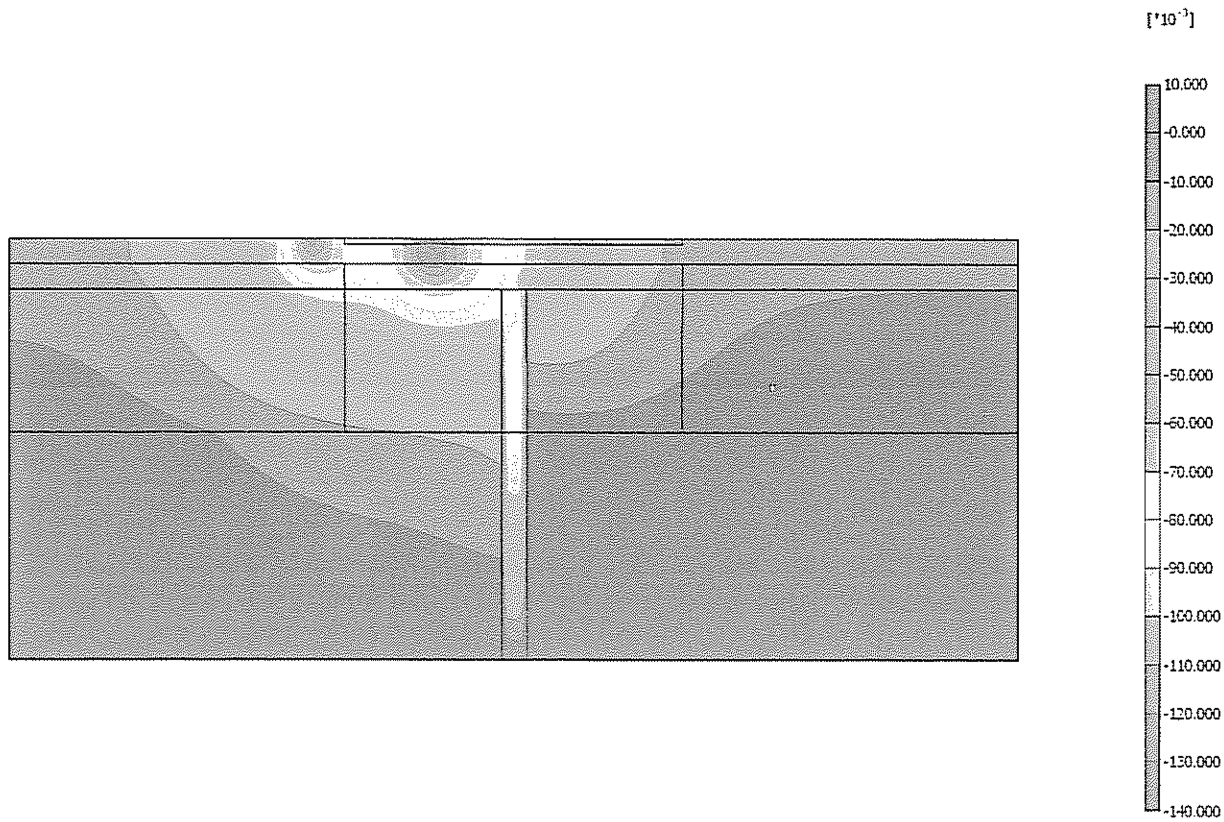

Fig. 19 Plot of vertical displacements (shadings)
- Step no: 236 - (Phase: 6)

7.9. Structures

7.9.11. Beams

7.9.11.3. Beams

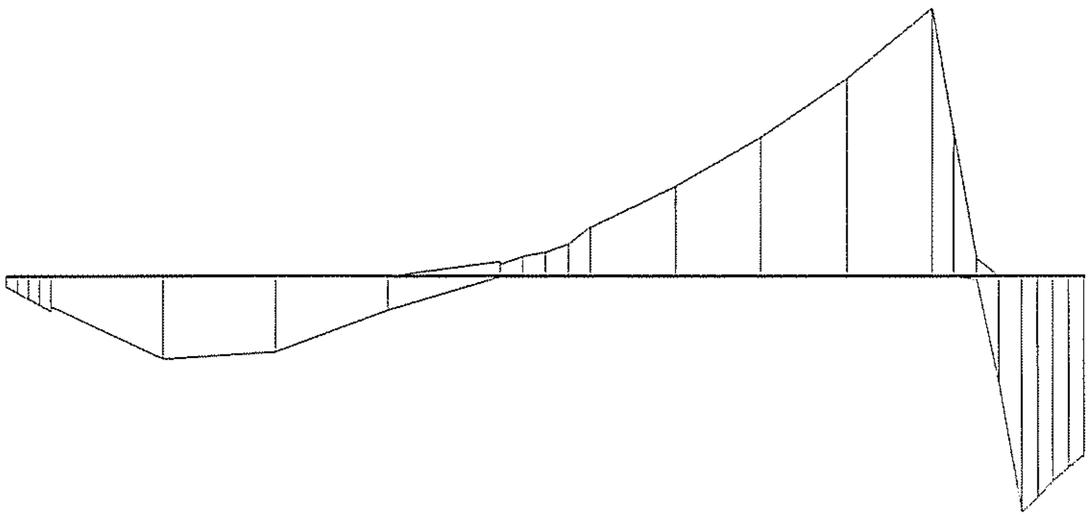

Fig. 20 Shear forces envelop in beam (plate no: 2)
Extreme value -739.38 lb/ft (Phase: 6)

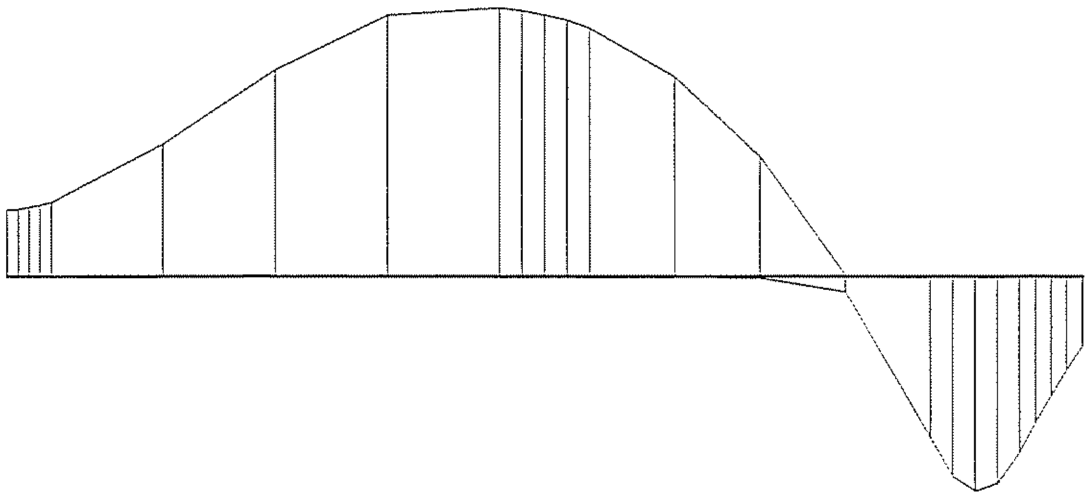

Fig. 21 Bending moment envelop in beam (plate no: 2)
Extreme value 833.76 lb/ft/ft (Phase: 6)

7.10. Cross sections

7.10.12. Deformations

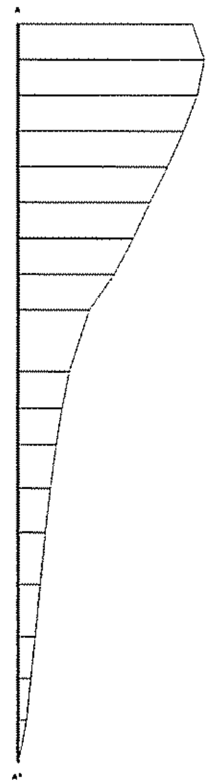

Fig. 22 Horizontal displacements in cross section (Cross Section A - A*)
Extreme value 24.00*10^{-3} ft (Phase: 6)

REPORT

05/20/2009

User: GeoSyntec Consultants

Title: 20-ft long concrete slab

Table of Contents

1. General Information

Table [1] Model

Model	Plane Strain
Element	15-Noded

2. Geometry

2.1. Clusters

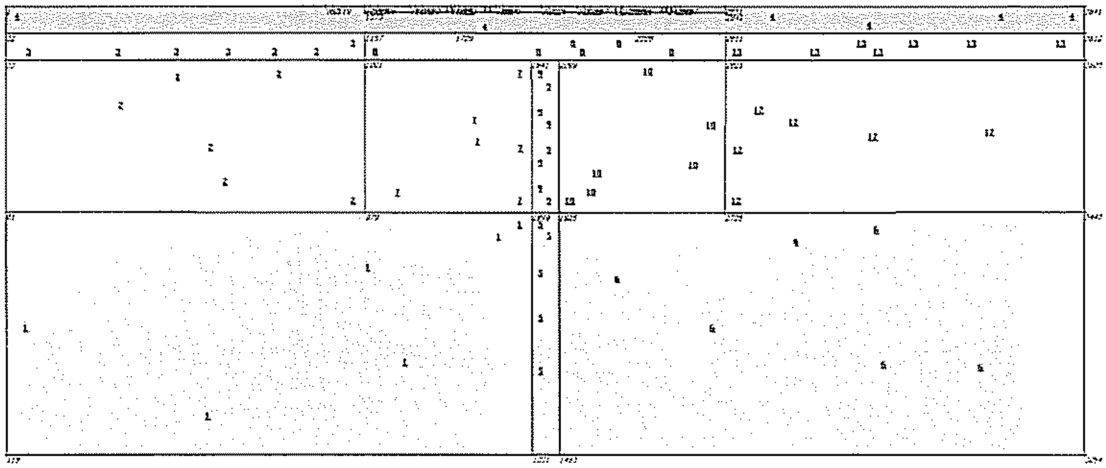

Fig. 1 Plot of geometry model with cluster numbers

3. Mesh data

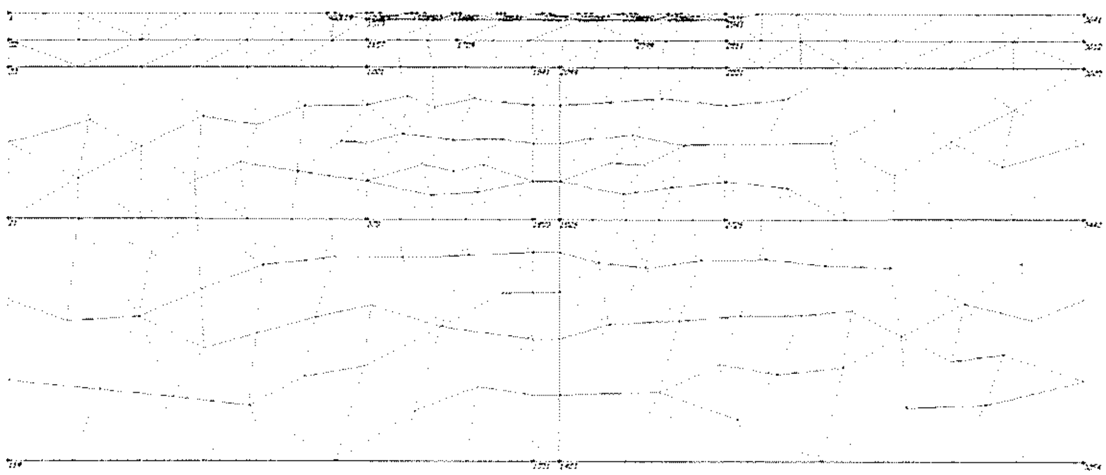

Fig. 2 Plot of the mesh with significant nodes

4. Material data

Table [2] Soil data sets parameters

Mohr-Coulomb		**1 Soil-Bentonite Backfill**
Type		Drained
γ_{unsat}	[lb/ft³]	105.00
γ_{sat}	[lb/ft³]	105.00
k_x	[ft/day]	0.000
k_y	[ft/day]	0.000
e_{init}	[-]	0.500
c_k	[-]	1E15
E_{ref}	[lb/ft²]	4200.000
ν	[-]	0.200
G_{ref}	[lb/ft²]	1750.000
E_{oed}	[lb/ft²]	4666.667
c_{ref}	[lb/ft²]	20.00
φ	[°]	0.50
ψ	[°]	0.00
E_{inc}	[lb/ft²/ft]	0.00
y_{ref}	[ft]	0.000
$c_{increment}$	[lb/ft²/ft]	0.00
$T_{str.}$	[lb/ft²]	0.00
$R_{inter.}$	[-]	1.00
Interface permeability		Neutral

Hardening Soil		**2 granular fill**	**3 refuse**	**4 Qrc**
Type		Drained	Drained	Drained
γ_{unsat}	[lb/ft³]	120.00	73.00	94.00
γ_{sat}	[lb/ft³]	120.00	73.00	94.00
k_x	[ft/day]	0.000	0.000	0.000
k_y	[ft/day]	0.000	0.000	0.000
e_{init}	[-]	0.50	0.50	0.50
e_{min}	[-]	0.00	0.00	0.00
e_{max}	[-]	999.00	999.00	999.00
c_k	[-]	1E15	1E15	1E15
E_{50}^{ref}	[lb/ft²]	700000.00	70000.00	34000.00
E_{oed}^{ref}	[lb/ft²]	560000.00	56000.00	27000.00
power (m)	[-]	0.50	0.45	0.85
c_{ref}	[lb/ft²]	2.00	5.00	5.00
φ	[°]	34.00	28.00	36.00
ψ	[°]	0.00	0.00	0.00
E_{ur}^{ref}	[lb/ft²]	2100000.00	157000.00	68000.00
$\nu_{ur}^{(nu)}$	[-]	0.200	0.200	0.200
p^{ref}	[lb/ft²]	2116.00	2116.00	100.00
$c_{increment}$	[lb/ft²]	0.00	0.00	0.00
y_{ref}	[ft]	0.00	0.00	0.00
R_f	[-]	0.90	0.90	0.90
$T_{str.}$	[lb/ft²]	0.00	0.00	0.00
R_{inter}	[-]	1.00	1.00	1.00

Hardening Soil	2 **granular fill**	3 **refuse**	4 **Qrc**
δ_{inter} [ft]	0.00	0.00	0.00
Interface permeability	Neutral	Neutral	Neutral

Table [3] Beam data sets parameters

no.	Identification	EA [lb/ft]	EI [lbft²/ft]	w [lb/ft/ft]	ν [-]	Mp [lbft/ft]	Np [lb/ft]
1	Concrete Plate	6.72E8	7.3E7	175.00	0.20	1E15	1E15
2	pavement	3.6E6	75000.00	60.00	0.20	1E15	1E15

5. Calculation phases

Table [4] List of phases

Phase	Ph-No.	Start phase	Calculation type	Load input	First step	Last step
Initial phase	0	0		-	0	0
Slurry Wall installation	1	0	Plastic analysis	Staged construction	1	114
Surcharge	2	1	Plastic analysis	Staged construction	115	130
surcharge removed	3	2	Plastic analysis	Staged construction	131	166
Load Position A	4	3	Plastic analysis	Staged construction	167	183
Load Position B	5	3	Plastic analysis	Staged construction	184	198
Load Position C	6	3	Plastic analysis	Staged construction	199	215

6. Results for phase 4

6.2. Deformations

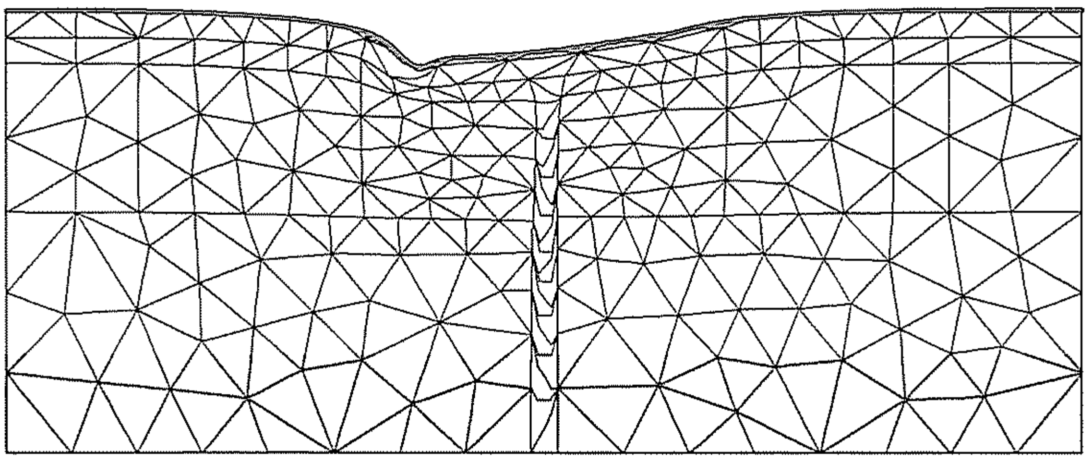

Fig. 3 Plot of deformed mesh
- Step no: 183 - (Phase: 4)

6.2.1. Plot of total displacements

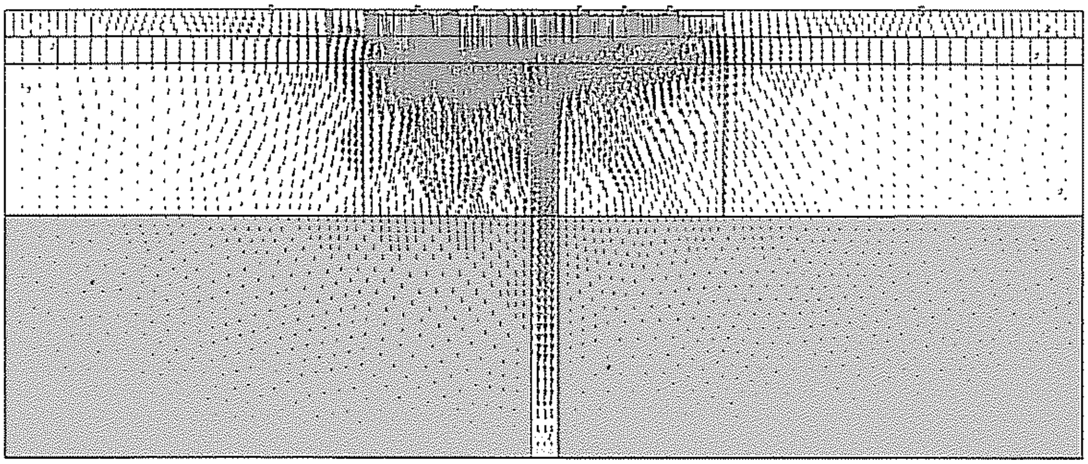

Fig. 4 Plot of total displacements (arrows)
- Step no: 183 - (Phase: 4)

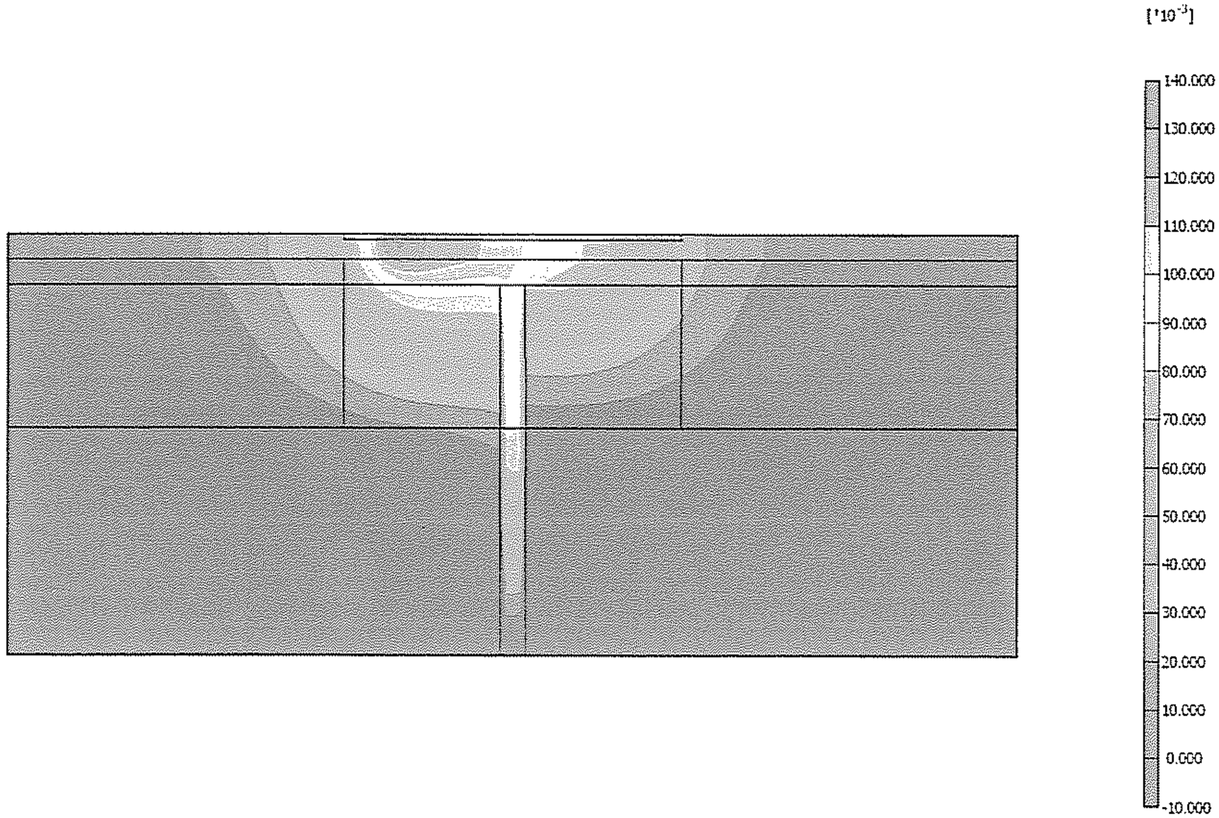

Fig. 5 Plot of total displacements (shadings)
- Step no: 183 - (Phase: 4)

6.2.2. Plot of vertical displacements

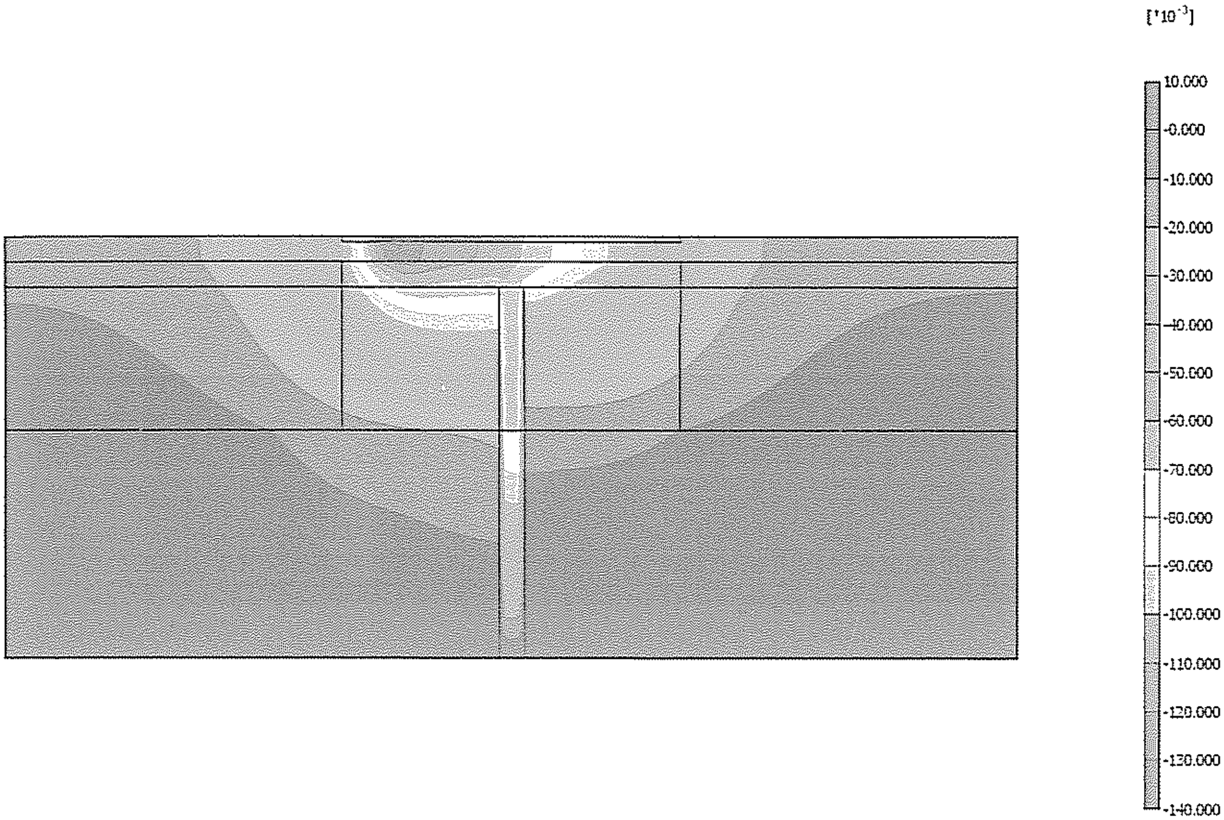

Fig. 6 Plot of vertical displacements (shadings)
- Step no: 183 - (Phase: 4)

6.3. Structures

6.3.3. Beams

6.3.3.1. Beams

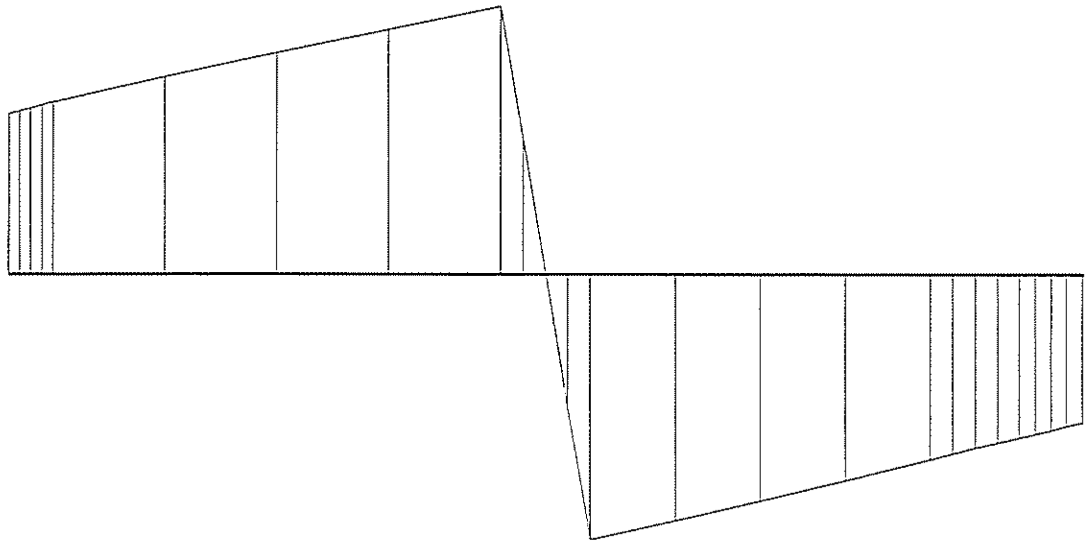

Fig. 7 Shear forces in beam (plate no: 2)
Extreme value $-3.11*10^3$ lb/ft (Phase: 4)

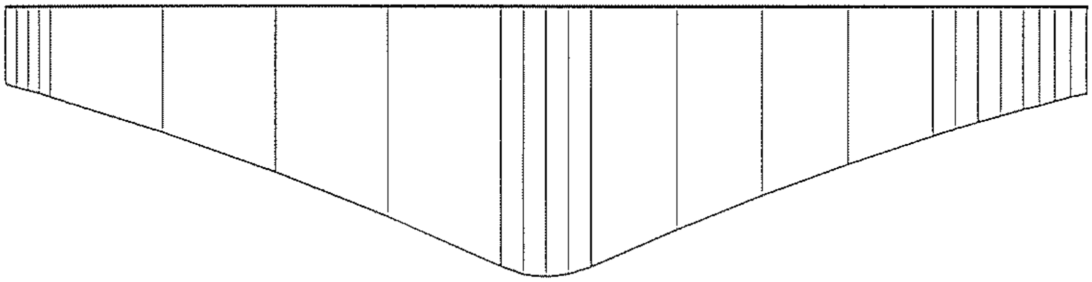

Fig. 8 Bending moments in beam (plate no: 2)
Extreme value $-17.02*10^3$ lbft/ft (Phase: 4)

6.4. Cross sections

6.4.4. Deformations

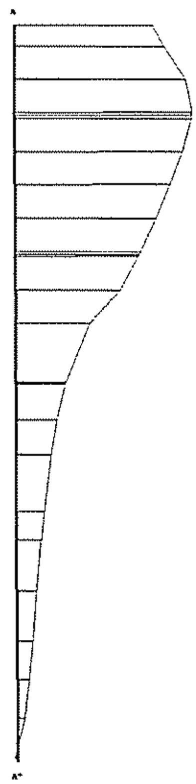

Fig. 9 Horizontal displacements in cross section (Cross Section A - A*)
Extreme value $12.99*10^{-3}$ ft (Phase: 4)

7. Results for phase 5

7.5. Deformations

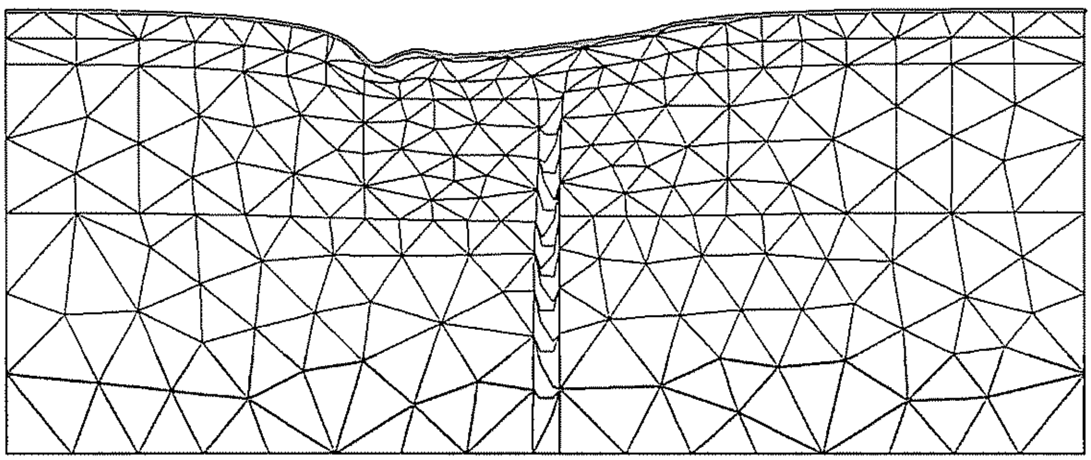

Fig. 10 Plot of deformed mesh
- Step no: 198 - (Phase: 5)

7.5.5. Plot of total displacements

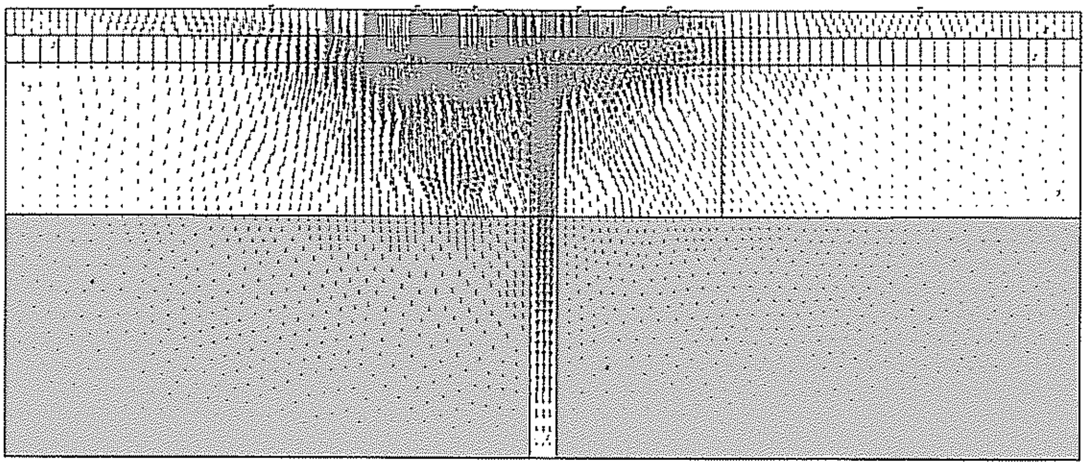

Fig. 11 Plot of total displacements (arrows)
- Step no: 198 - (Phase: 5)

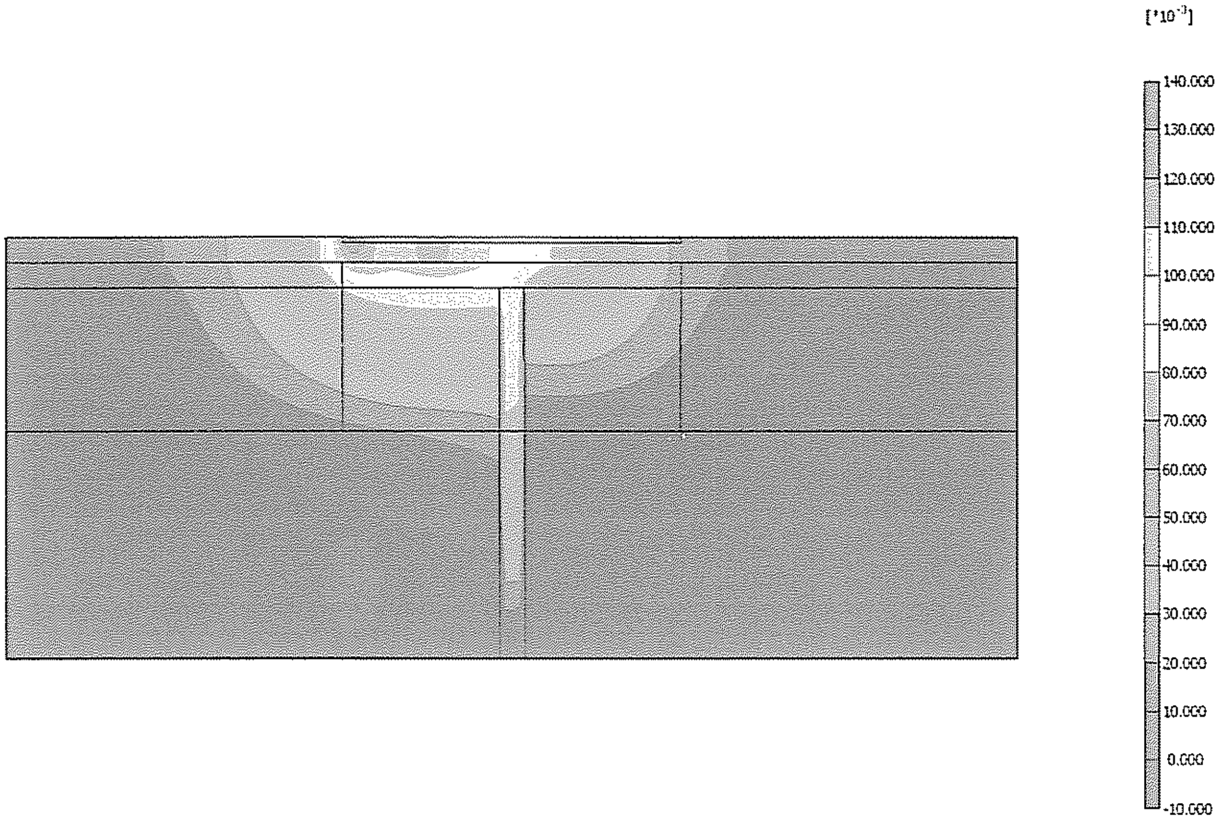

Fig. 12 Plot of total displacements (shadings)
- Step no: 198 - (Phase: 5)

7.5.6. Plot of vertical displacements

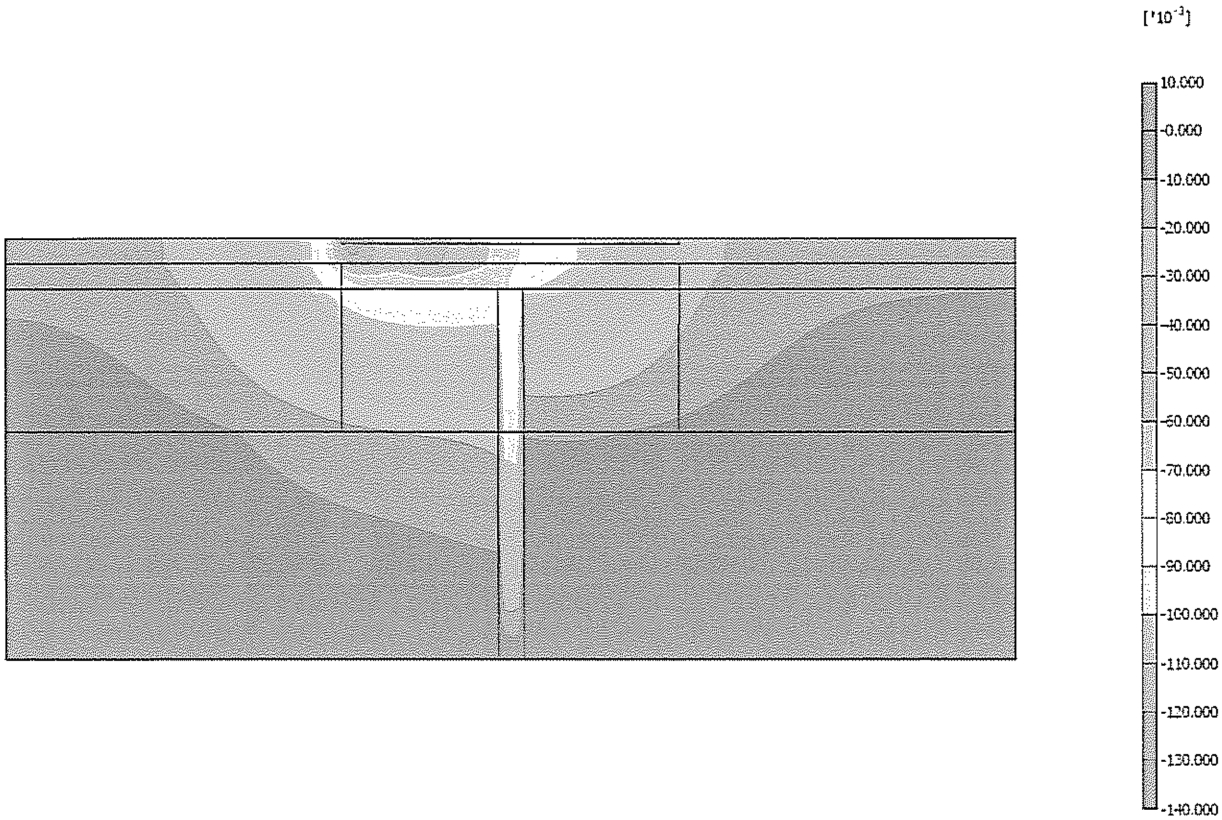

Fig. 13 Plot of vertical displacements (shadings)
- Step no: 198 - (Phase: 5)

7.6. Structures

7.6.7. Beams

7.6.7.2. Beams

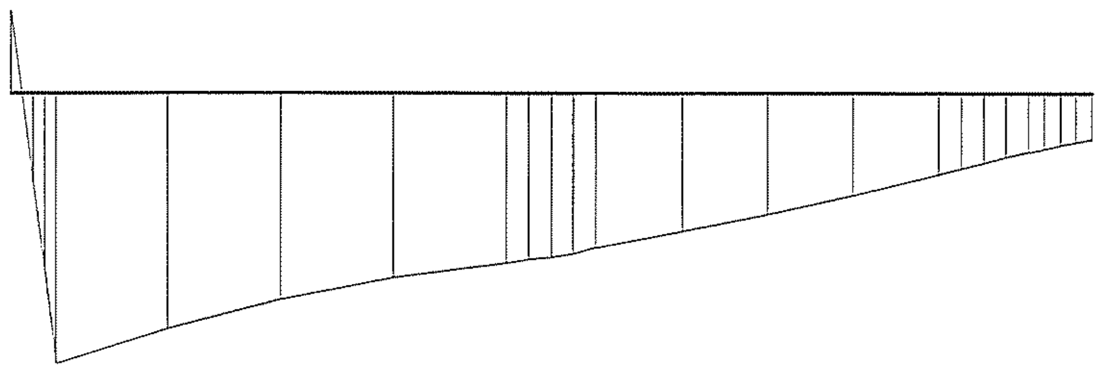

Fig. 14 Shear forces in beam (plate no: 2)
Extreme value $2.41 * 10^3$ lb/ft (Phase: 5)

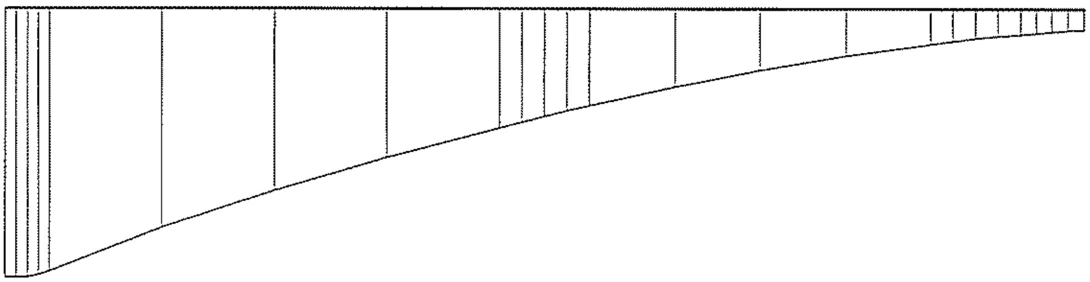

Fig. 15 Bending moments in beam (plate no: 2)
Extreme value $-14.72*10^3$ lbft/ft (Phase: 5)

7.7. Cross sections

7.7.8. Deformations

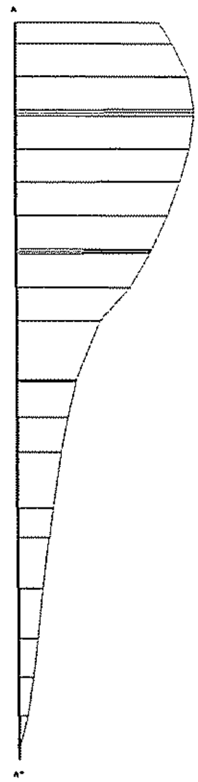

Fig. 16 Horizontal displacements in cross section (Cross Section A - A*)
Extreme value 15.17*10^{-3} ft (Phase: 5)

8. Results for phase 6

8.8. Deformations

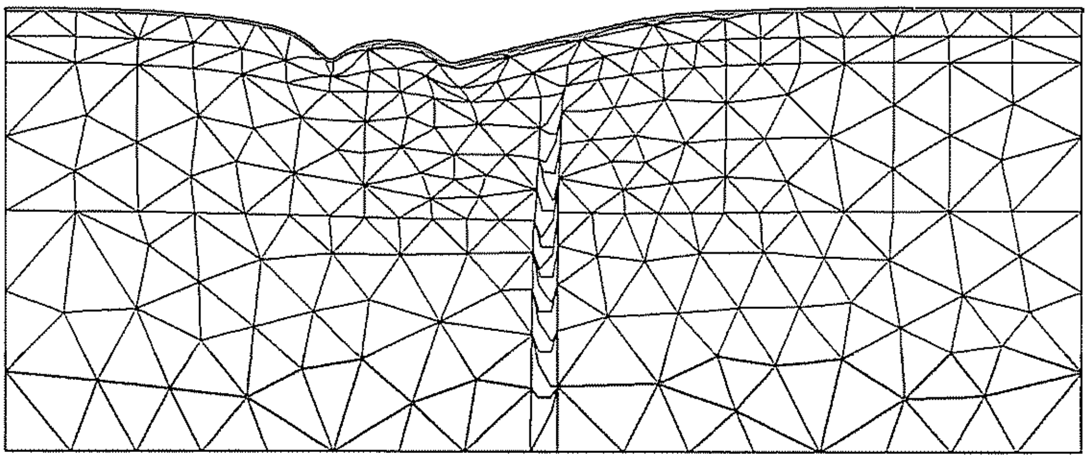

Fig. 17 Plot of deformed mesh
- Step no: 215 - (Phase: 6)

8.8.9. Plot of total displacements

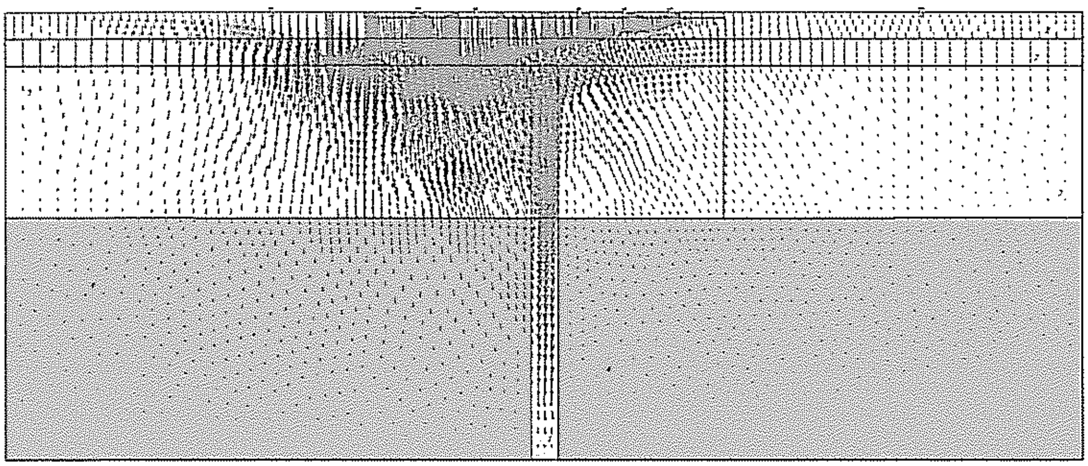

Fig. 18 Plot of total displacements (arrows)
- Step no: 215 - (Phase: 6)

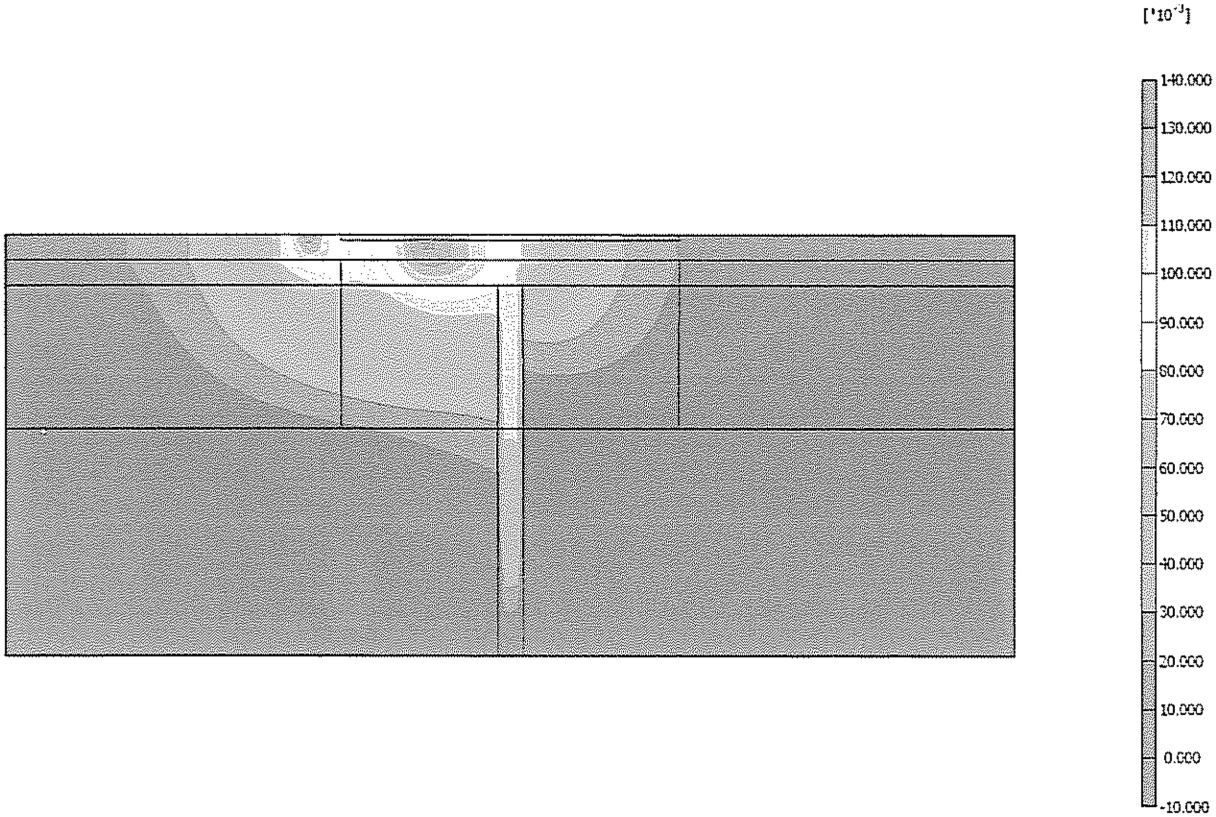

Fig. 19 Plot of total displacements (shadings)
- Step no: 215 - (Phase: 6)

8.8.10. Plot of vertical displacements

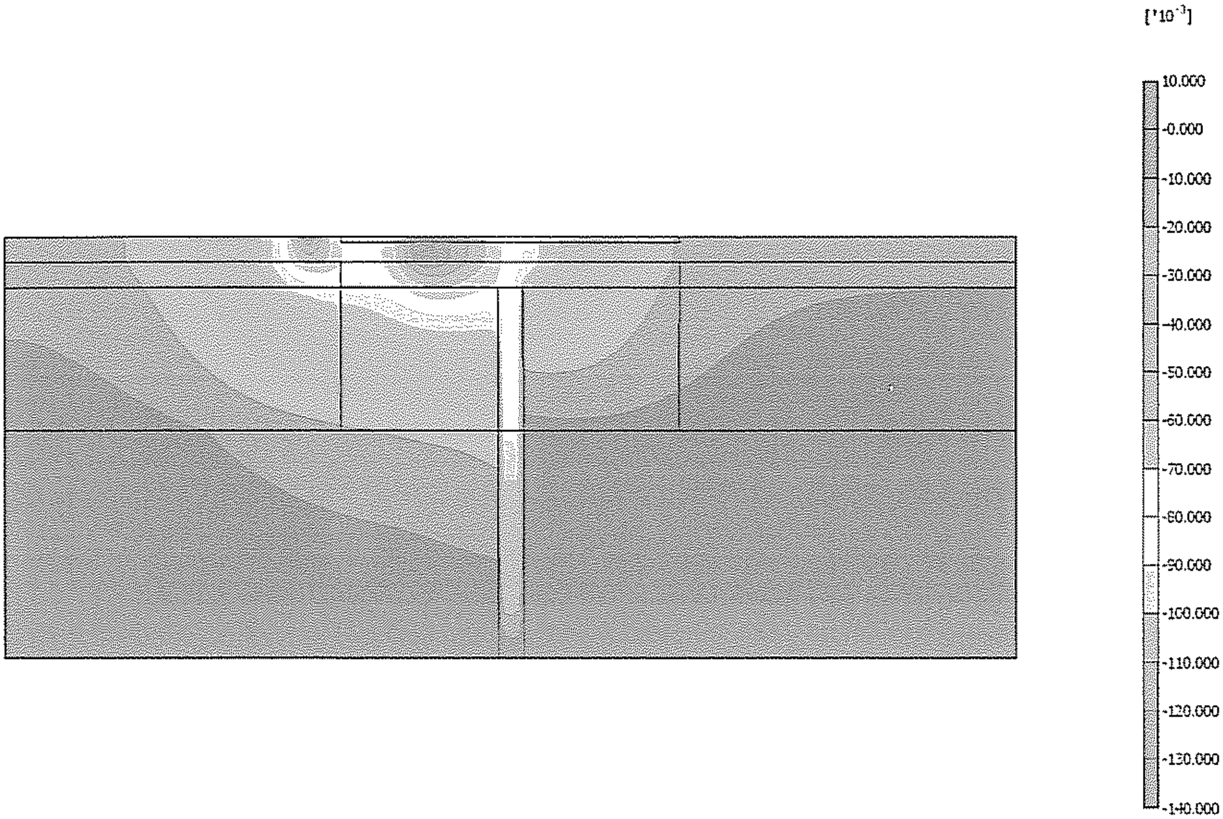

Fig. 20 Plot of vertical displacements (shadings)
- Step no: 215 - (Phase: 6)

8.9. Structures

8.9.11. Beams

8.9.11.3. Beams

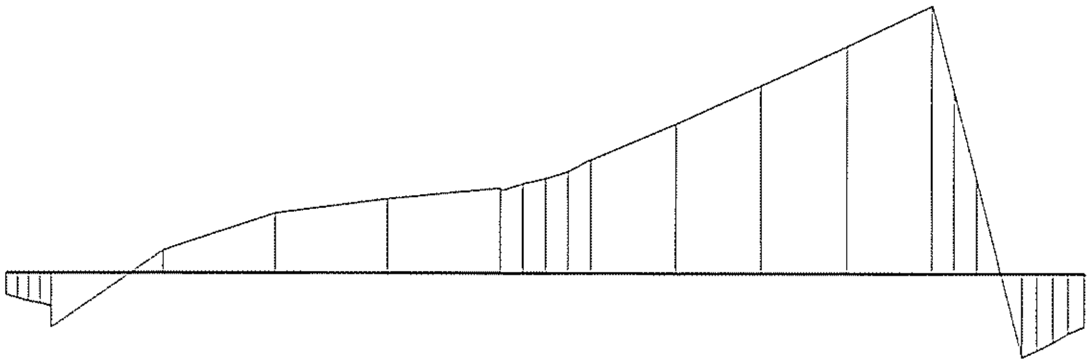

Fig. 21 Shear forces in beam (plate no: 2)
Extreme value $-1.09*10^3$ lb/ft (Phase: 6)

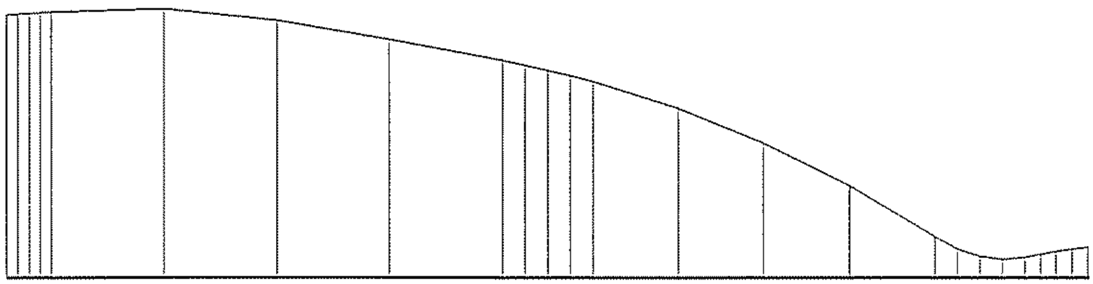

Fig. 22 Bending moments in beam (plate no: 2)
Extreme value $4.21*10^3$ lbft/ft (Phase: 6)

8.10. Cross sections

8.10.12. Deformations

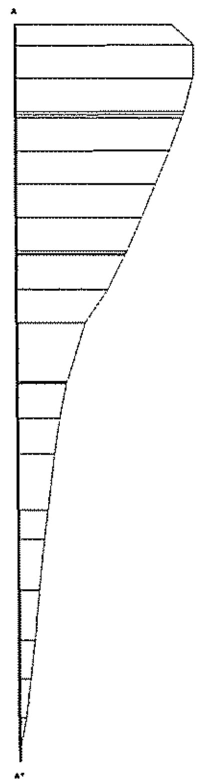

Fig. 23 Horizontal displacements in cross section (Cross Section A - A*)
Extreme value $22.98*10^{-3}$ ft (Phase: 6)