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The Behavior and the Response of Buried Circular Metallic Pipe under Embankment

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Abstract: This study treats on soil-structure interaction in the case of circular metal conduits buried in an elastic soil. The use of Finite Element Method (FEM) proves to be necessary to evaluate the most used theory, namely the empirical Marston and Spangler theory. For this purpose, computation sofware baed on the FEM was developed to highlight the response of such structures in terms of the influence of different parameters such as the ratio H/D (diameter on the height cover), the ratio D/t (diameter on conduit thikhness) and the soil properties constituting the embankment illustrated by the ratio E_s/E_g (modulus of elasticity of the subgrade or substratum). All the parameters were studied according to two methods of staged on sequential construction. In this study a qualitative and quantitative comparison of results obtained by the two methods is undertaken. This study highlighted the rliability and power of FEM compared to the empirical Marston Spangler method as well as thneed for considering the history of the construction of the solid soil structure.

Key words: Soil, structure, interaction, pipe, conduit, buried, behavior, response, embankment

INTRODUCTION

A detailed report on the state of knowledge is presented by Krizek *et al*. (1971). The reports examines past and present researches, designe procedures and field performance related to pipe culverts and recommends the use of empirical methods of load and deflection calculations for the majority of cases.

Marston developed the expressions for the total vertical loads over the pipe o be applied without considering the detailed distributions of interfacial stresses over the pipe surface. These calculations are mainly based on settlement ratio r', projection ratio and location of Plane of Equal Settlement (PES). The factors selected on this basis are obtained from experiments and are quite arbitrary. Marston's load theory some than seventy years ago forms the basis for a pipe design till now without substantial change. However, Spangler (1941) has modified this quasi elastic approach of equating settlements at PES due to weight of soil of the total height of fill. M.G. Spangler has also given horizontal soil pressures depending upon the modulus of passive resistance of the fill material. This theory is called now the Marston Spangler theory.
The load W_C is given by

$$W_C = C_c \gamma D^2 \qquad (1)$$

γ is the unit weight of the fill and C_c is the load coefficient which depends upon mobilisation of shear stresses on the prism. It is therefore, necessary to assume suitable interaction between the interior prism and the exterior prisms of some suitable width and vice versa, depending upon the relative movement between the prisms.The transfer load has been assumed to depends upon un abstact number called settlement ratio r', of the critical plane at the crown and the adjoining fill. It is defined as

$$r' = [(S_m + S_g) - (S_f + \Delta V)] / S_m \qquad (2)$$

Where S_m, S_g and S_f settlements of the soil in projection height pD, natural ground and invert and ΔV is the vertical shortning of pipe diameter Fig. 1 and 2.

Brown (1967) has analysed a rigid culvert of horse shoe shape by using constant strain triangular elements in plain strain (Brown *et al*., 1968; Brown and King, 1966). The effect of sequential placement of layers with the inclusion of hay material in the interior prism has been reported. Brown *et al*. (1968) have reported that the induced trench condition for flexible pipe, reduces the crown pressure and increase the side pressures.The use of suitable element to represent pipe behavior has been emphasised.

Abel *et al*. (1973) have studied the effect of pipe stiffness, depth of cover and the effect of slippage at the interface for an elliptical pipe for surcharge loading. The pipe has been placed in uniform soil all around. It is found that the effect of Poisson ratio of pipe material and the soil is negligible and the slip at the interface is beneficial for establishing arching action, particulary for deeply buried

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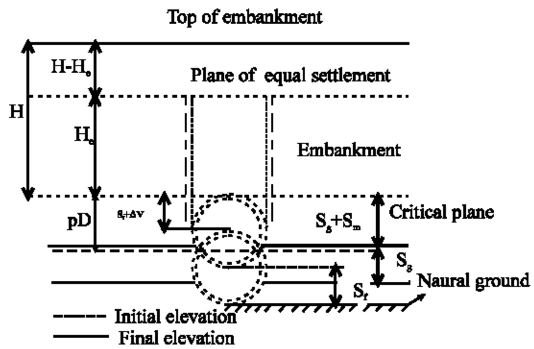

Fig. 1: Rigid conduit

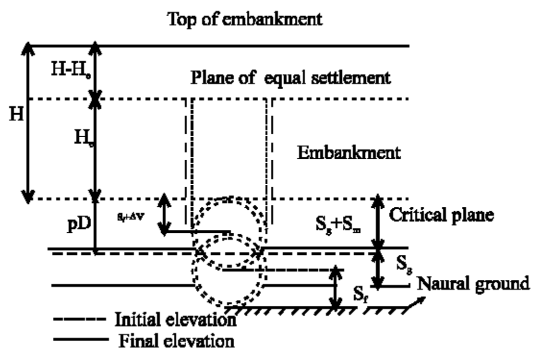

Fig. 2: Rigid conduit

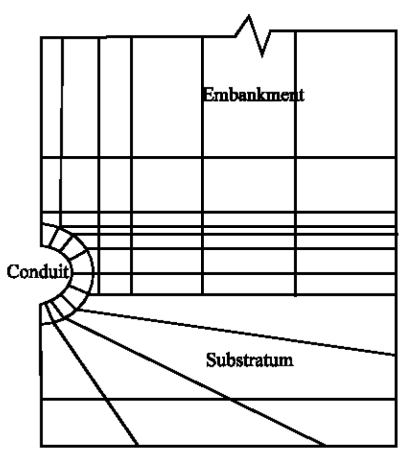

Fig. 3: Finite element idealization

rigid pipes. Anand (1974) presented results for shallow buried rigid pipes due to superimposed surface loads. For linerar elastic soil properties he has found the effect of uniformly distributed load of various length.

Darwin (1983) and Chang *et al.* (1980) have shown the importance of effect of sequential placement of layers by using some experimentations on rigid pipes and incremental construction phases study and process of embankment construction (Fig. 3).

Hoeg (1968) has examinated in detail the magnitude and the distribution of static normal stresses against horizontal buried cylinders in homogeneous deposit of dry sand. The experimental results, obtained from laboratory tests employing a technique for measuring contact stresses have been presented. The results show the influence of structural flexibility and compressibility and depth of soil cover. Analytical predictions have been made using idealized mathematical model and theory of elasticity to identify the significant variables and to examine their effects. It is found that Analytical analysis mathematical model and theory of elasticity the experimentally measured results have shown the influence of flexibility and compessibilty of the structure as well as the effect of cover height variation on it.

White and Layer (1960) have developped a simple process to design approach based on ring compression theory which is applied for designing some structures. The theory stipulate that the culvert supports the embankment weight on crown by ring compression only. However, bending moment could be developped with plastic hinge appearance on the culvert wall. Nevertheless, the deflections remains small, in case of buried structure in well controlled backfilling.

Sidney and David (1988) have shown that the final deformations and stresses in buried structures that interact with the surrounding soil are construction dependent.

They have also considered the modelling of construction steps and have proposed a technique to overcome some model construction problems such numerical instability of the solution.

Karinski *et al.* (2003) have analysed a discrete-continuous model to study a buried structure response to static surface loading as well as the soil gravitational load at'service-state'conditions is presented. A two-degree-of-freedom model represents the structure above which a continuous vertical column represents the soil. The proposed model simulates the soil-buried structure interaction affected by the structure's roof displacement as well as the rigid body displacement of the whole structure relative to that of the free field. The model can represent positive and negative arching and provides an understanding of the effects that various variables have on the arching type and on the structure response. Other soil-structure parameters that are included in the model are the soil and structure material properties, roof span and thickness, the structure's heigh and the depth of burial and external pressure. Simulations of a rectangular buried conduit performed by both the proposed model and by a finite element analysis yielded similar interface loads and similar influence of the problem parameters on the results. This example demonstrates the effect of the structure's stiffness and height on the soil arching above it and on the average interface load acting on its roof. Thus, the proposed model can be used in preliminary stages of the design process to easily evaluate the effect of variables such as the structure properties on the response.

MATERIALS AND METHODS

Single lift linear elastic analysis: The behavior of steel pipe 1 m. in diameter and thickness corresponding to various D/t ratios, has been studied in plane strain condition for backfill loading. The bedding angle φ of 37.5° corresponding to projection ratio p = 0.9 has been selected for all the cases. The conduit material properties have been taken as E_{st} = 2.1*10^6 kg cm^{-2}, ν = 0.3 and γ = 8.0 g cm^{-3}.

Effect of H/D ratio: Five cases of height of fill with different H/D ratios of 1, 2, 3, 4.5 and 6.5 are analysed for gravity loading in a single lift. The backfill is assumed to consist of elastic homogeneous soil of E_s = 300 kg cm^{-2}, ν = 0.3 and γ = 1.5 g cm^{-3}.The thickness t of the pipe has been taken as 1 cm (D/t = 100) and the ground is considered to be stiff with material constants E_g = 3000 kg cm^{-2}, ν = 0.3.

The noniniform normal contact pressure σ_n is plotted in Fig. 4. The pressure σ_n increases with H/D ratio all along the conduit surface. At the shoulders (45° from crown) σ_n is lower than that at the crown. The maximum value of σ_n occurs in the area where the conduit meets the ground due to an outward relative movement and it mobilises pressure nearer to passive earth pressure. The shear stresses at the interface τ_n is plotted in Fig. 5. The shear stress become zero at about 100° from the crown where it changes sign from negative (downward fill action) to positive. ratios.

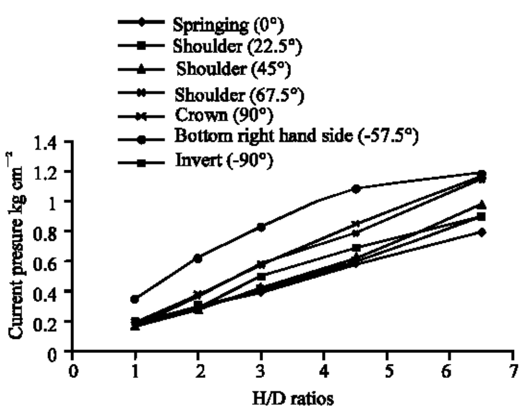

Fig. 4: Contact pressure distibution for various H/D

The total load/unit length, obtained by integrating at top of the conduit, is compared with WC obtained by Marston's load theory. The values of W_C for various settlement ratios obtained by FEM are given in Table 1 for both the complete and incomplete projection conditions. The settlement ratio r' is found to decrease with increase in H/D ratio, keeping all other parameters to be inchanged. Spangler recommended values of r' to lie between 0.5 and 0.8 irrespective of dept of fill. The value obtained by elastic analysis lie between 0.42 and 0.52.

Table 1 shows that WC calculated at critical plane level (neglecting the effect of load due to haunches) according to Marston load theory for incomplete projection condition is about 19% higher than W_F calculated by taking the haunch load into account. The PES for elastic analysis for all H/D ratios does not lie within the embankment and therefore the loads calculated on the basis of complete projection condition for all cases are even higer than (Table 1). According to Spangler the horizontal stress is dependant on the modulus of soil reaction.

$$E' = \sigma_h * D / \Delta H \tag{3}$$

σ_h is the spring line pressure and Δhorizontal diameter. Since no clear guidelines are available for the determination of E', the ratio E'/E_s decrease with H/D ratio and its average value for the pipe of D/t = 100 and E_s/ E_g = 0.1 is about 2.1.

Effect of D/t ratio: Three cases of wall thikness with D/t ratios of 40, 100 and 400 are analysed by keeping H/D = 6.5,
E_s = 600 kg cm^{-2} and E_g = 3000 kg cm^{-2} constant.The contact pressure distribution is shown in Fig. 6.

Table 1: Wc at critical plane level

H/D	W_C en kg cm^{-1} Marst-spang proj -comp	Marst-spang proj -incomp	W_F en kg cm^{-1} M.E.F	((5-3)/3)*100	E'/ E_s	r'
1	2	3	4	5	6	7
1.0	1827	2004	2004	9.69	2.34	0.524
2.0	4500	4275	3670	-14.15	2.24	0.477
3.0	14415	6450	5420	-15.97	2.19	0.460
4.5	17820	9750	7720	-20.82	2.17	0.454
6.5	42600	13800	10900	-21.01	2.12	0.427

As shown in Table 2 the crown pressure increase and springing line pressure decrease with increase in wall thikness. The pipe with D/t = 400 behaves as flexible pipe. The rigid conduit with D/t = 40 mobilises much higher σat the ground level.

The magnitude of shear stress and average τ_n/ σ_n ratios, shown in Fig. 7 increases with increase in D/t ratio.

Table 2 shows Marston's load theory (incomplete projecting condition) yielding 16% higher load for flexible pipe with D/t = 400. The E'/E_s ratio decrease with increase in D/t ratio.

Effect of E_s/E_g ratio: Similar analysis are carried out for three backfill materials with $E_s$variying as 300, 600 and 900 kg $cm^{-2}$and E_s / E_g = 0.1, 0.2 and 0.3 for the pipe with D/t = 100 and fill loading corresponding to H/D = 6.5. The contact pressure σ_n is plotted in Fig 5. The load on the pipe according to Marston's theory (incomplete projecting condition) and E' /E_s tabulated in Table 3.

The contact pressures σ_n and shear stresses τ_n as shown, respectively in Fig. 8 and 9 for three cases of backfill are marginally affected.

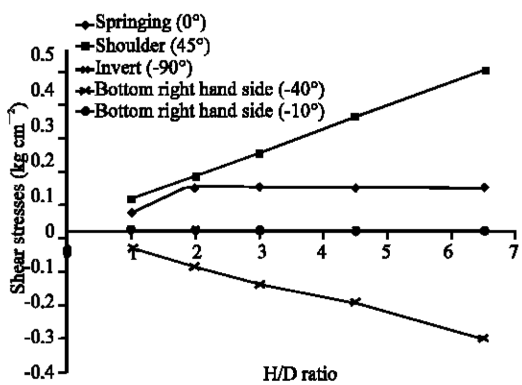

Fig. 5: Variation of shear stresses at the interface for various H/D ratios

The results of three more analyses for E_s / E_g = 1 (homogeneous condition) and pipe with D/t = 400 are presented in Table 4 for fill loading corresponding to H/D = 6.5. It shows how the response of the pipe with D/t = 400 changes from rigid to flexible condition.

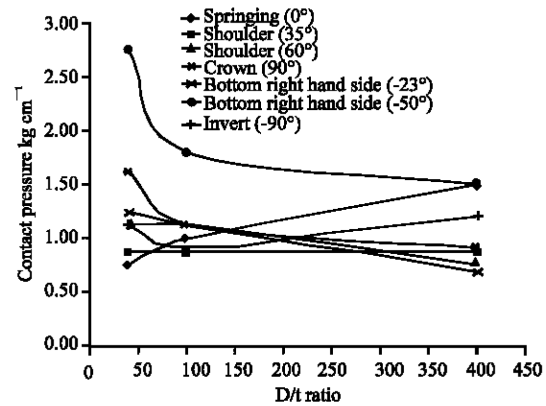

Fig. 6: Contact presure distribution for various D/t ratio

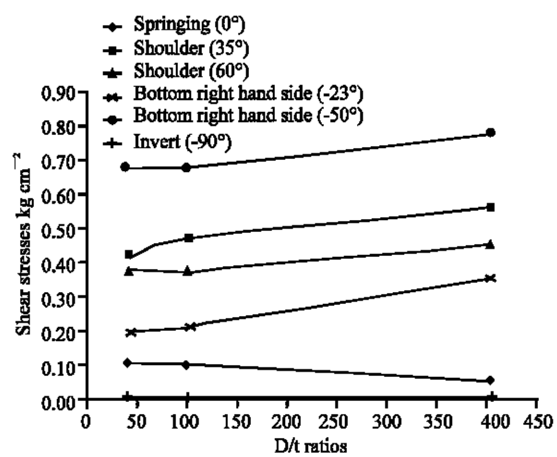

Fig. 7: Variation of shear stresses for various D/t ratios

Table 2: The E' /E_s ratio decrease with increase in D/t ratio

	σ_n / γ H				W kg m^{-1}			
D/t	Crown	Springing	Invert	r'	Marst-spangler	F.E.M	((6-7)/6)*100	E'/E_S
1	2	3	4	5	6	7	8	9
40	1.260	0.747	1.170	0.470	13950	11400	18.3	2.38
100	1.080	0.933	0.959	0.292	12750	10900	14.5	1.90
400	0.714	1.480	0.949	-0.200	7800	9500	-21.8	1.59

Table 3: The load on the pipe according to Marston's theory

	σ_n / γ H				W kg m^{-1}			
D/t	Crown	Springing	Invert	r'	Marst-spangler kg cm^{-1}	F.E.M kg cm^{-1}	((6-7)/6)*100	E'/E_S
1		3	4	5	6	7	8	9
0.1	1.16	0.810	0.935	0.427	138.0	107.6.0	22.03	2.12
0.2	1.05	0.930	0.953	0.292	127.5	109.5.0	14.11	1.90
0.3	0.992	0.970	0.978	0.179	126.0	111.500	11.50	1.78

Table 4: Analyses for E_s/E_g = 1 and pipe with D/t = 400

	σ_n / γ H				W kg m^{-1}			
D/t	Crown	Springing	Invert	r'	Marst-spangler Wc kg cm^{-1}	F.E.M W_F kg cm^{-1}	((6-7)/6)*100	E'/E_S
1		3	4	5	6	7	8	9
0.2	1.74	1.10	0.650	0.705	147.0	103.0	29.31	1.70
0.5	0.985	1.25	0.900	0.049	108.0	100.0	7.40	1.58
1.0	0.612	1.38	0.978	- 0.026	93.0	97.9	- 5.26	1.60

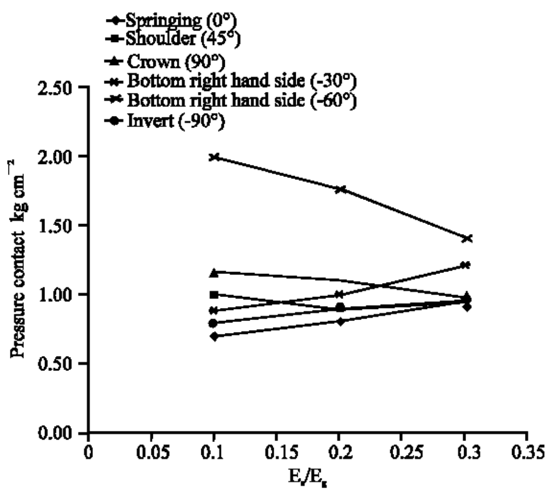

Fig. 8: Pressure contact distribution for various Es/Eg

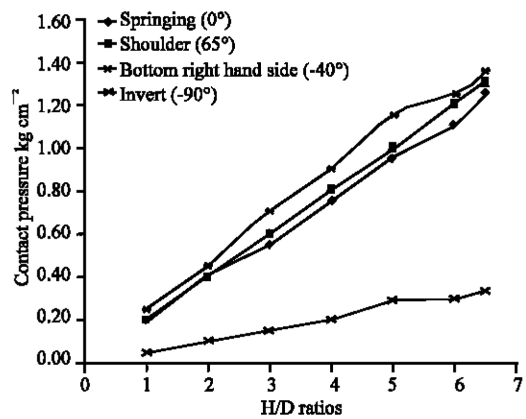

Fig. 10: Contact pressure distribution for linear sequentiel analysis

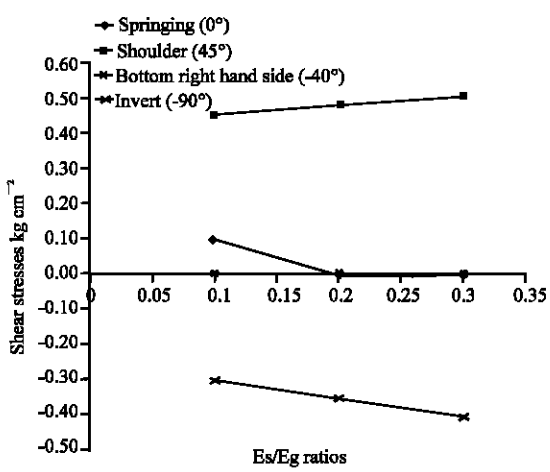

Fig. 9: Variation of shear stresses for various Es/Eg ratios

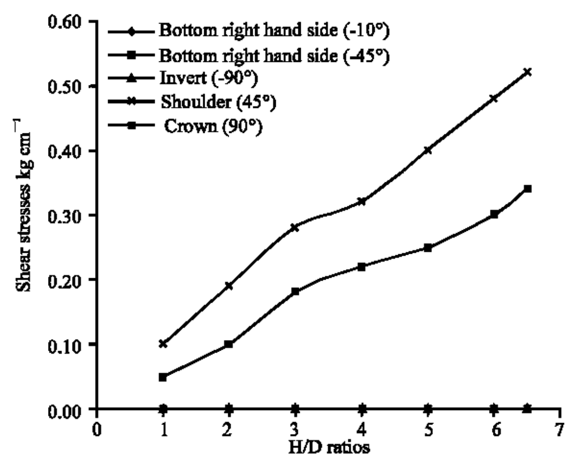

Fig. 11: Shear stressesdistribution for various H/D ratios for linear sequential anysis

RESULTS AND DISCUSSION

In order to find the effect of placement of backfill in layers one typical case with linear material is considered and analysed in wich the pipe having D/ t = 100 has been placed on stiff ground (E_g=3000 kg cm^{-2}, ν = 0.3) and is backfilled with soil upto H = 6.5 m in seven layers. The first 6 layers are of one meter height and the last one of 0.5 m.

The actual sequence of placement of backfill is modeled in program. Although the total fill loading acting in the system is the same as in the case of single light analysis, the defomation patterns and soil-pipe interaction are quite different.The modification of contact pressure distributions for one case with linear behavior is, therefore, briefly presented in Fig. 10.

Table 5: The values Wc W_F and the percentage difference

	$\sigma_{cr}/\gamma H$		$\sigma_h/\gamma H$		W kg m^{-1}LS		
H/D	SL	SL	SL	SL	Marst-Spangler W_C kg cm^{-1}	F.E.M W_F kg cm^1	H/D((6-7)/6)*100
1	2	3	4	5	6	7	8
1.0	1.25	1.25	1.18	1.18	18.3	20.0	8.5
2.0	1.21	1.28	0.910	1.02	43.5	38.5	11.5
3.0	1.22	1.29	0.851	0.967	64.5	57.2	11.3
4.0	1.22	1.30	0.851	0.933	86.3	75.8	12.2
4.5	1.21	1.30	0.830	0.933	86.3	75.8	12.2
5.0	1.21	1.30	0.830	0.920	105.0	92.4	12.0
6..0	1.21	1.30	0.830	0.911	123.8	102.3	17.4
6.5	1.18	1.30	0.815	0.903	135.0	108.0	20.0

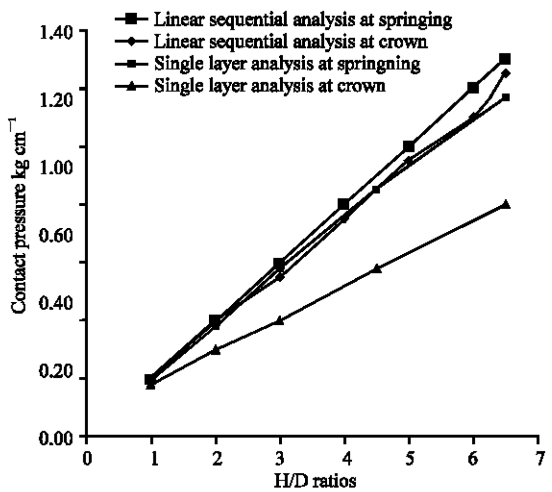

Fig. 12: Crown and springing contact pressure for various analysis

Linear sequential analysis: The analysis is carried out by assuming linear backfill properties as $E_s = 300$ kg cm^{-2}, $\nu = 0.3$ and $\gamma = 1.5$ g cm^{-3}. The normal contact pressure distributions σ_n and shear stress distributions τ_n all along the pipe surface for H/ D ratios of 1, 2, 3, 4, 5, 6 and 6.5 are plotted in Fig. 10 and 11. Qualitatively the variation of contact pressures are similar to linear single lift analysis (SL).There is in general increase in the normal pressures at the crown an springing as indicated in Table 5.

The load W_F due to placement of each layer is compared with WC (Marston's incomplete projection) by using the settlement ratios obtained from Linear Sequential analysis (LS). The values WC, W_F and the percentage difference is given in Table 5. The increase in W_F for Linear Sequential analysis (LS). Over linear Single Lift analysis (SL) is due to the fact that exterior column settles more in the case of Linear Sequential analysis (LS). As in Single Lift analysis (SL) the PES doesn't lie within embankment for all value of H/D ratios.

Comparison: A comparison between the results from linear single layer elasic analysis, linear sequential analysis is shown in Fig. 12 and Table 5 for the pipe of D/t = 100 for various H/D ratios.

CONCLUSION

The finite element method provides an appropriate numerical modelling of interaction problems of buried pipe. It can account for linear single lift analysis and sequential placement of backfill analysis and the actual field simulation can be done for any geometry of the system. The range study for parameters such as diameter thikness ratio of pipe, depth of fill diameter ratio and modulus of soil surrounding the pipe for the contact pressure distributions of pipe has been carried out in details. Qualitatively the variation of contact pressures are similar. There is in general increase in the normal pressures at the crown and springing both for both the cases with the placement of soil in layers (linear sequential analysis) and linear single lift analysis. The PES for linear single lift and linear sequential analyses does not lie within embankment for any height of fill. The critical examination of Marston-Spangler load theory has been made. The limitations of this theory have been clearly indicated. The design for large diameter pipe based on complete contact pressure distributions has been recommended

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