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Behavior of Buried Water Pipes under Loads and Factors Affecting it

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ABSTRACT

Water pipelines are also called as lifelines as they serve the most important parameter to the peoples. Hence water pipelines are most important structural element and to make it more safe take utmost priority. When pipelines are subjected to various loads such as earth fill or surcharge load, uplift pressure, lateral side pressure and sometimes may be subjected to superimposed traffic load if roadway or railway comes over it.The various factors such as depth of fill above pipe, side trench width, internal water pressure, support conditions are the some factor which are observed, which greatly affect he stress behavior in the pipeline. The study and experimental work done shows the satisfactory parameters for making underground pipe structure more safe and efficient and thus will prove beneficiary to society even in worse disasters such as earthquake.

Introduction

Generally water pipes may be flexible or rigid in nature. Flexible pipes don't have to support the extra loading. They deflect and transfer the load and the filling material and ground adjusts in support. The deflection of the pipe is a reaction to the settlement of the ground together with the pipe. This arrangement is stable and rigid enough to support the traffic loads and those from its own weight, and the filling material of the pipe installation. Flexible pipes primarily represented by corrugated metal and plastic pipe, etc.

Rigid pipes are always more strong and healthy, and rigid than the surrounding ground, therefore the loading is concentrated in the pipes. As rigid pipes can't be deformed, they have to be able to fully support the loads themselves, meaning the possibility of a break is higher. Rigid – represented primarily by reinforced concrete pipe but also includes plain concrete and clay pipe.

Although in this study we are dealing with rigid pipes i.e. only R.C.C pipes. Rigid pipe has significant inherent strength to support loads without aid from the backfill. However, its load-carrying capacity can be increased by a factor of 4 or more by taking special measures in bedding and backfilling the pipe. Rigid pipe fails by breaking or fracturing before deflections have become appreciably large. Unreinforced rigid pipe, such as clay or plain concrete, is commonly considered to be failed when it has cracked or fractured. However, in reinforced concrete pipe the appearance of a crack as wide as a 0.01 in. is commonly considered permissible. A crack, without collapse of the pipe, obviously does not prevent rigid pipe from serving as a conduit, yet it does represent a loss in strength of a non-reinforced pipe and possible exposure of the reinforcing steel by wide cracks in reinforced pipe.

Aim of the Study:

1)To study about the stresses, relation between hoop stresses and longitudinal stresses and design principles for underground pipeline**.**

2)To analyse the pipeline under external stresses to which it is subjected.

3)To develop the model for the same using STAAD-Pro.

Various factors affecting the behavior of rigid pipes:

a)Effect of fill height above pipe:

High fills increase the probability of dead load problems. As the fill height is increased, the weight of the soil supported by a buried pipe is likewise increased.

b)Importance of side support:

If the vertical diameter of a circle of fixed circumferences is shortened, its horizontal diameter is lengthened. It follows then that the support which tends to prevent the lengthening of the horizontal diameter of a circular pipe will resist loads tending to shorten its vertical diameter. Rigid pipe, while capable of carrying larger loads without side support, is limited by its inherent strength. The addition of side support increases its vertical load-carrying capacity. The need for side support, while more obvious and extremely important for flexible pipe, can be quite important for rigid pipe also.

c)Effect of wheel or traffic loads with fill depth:

As the depth of fill increases the intensity of wheel load effect or traffic decreases.

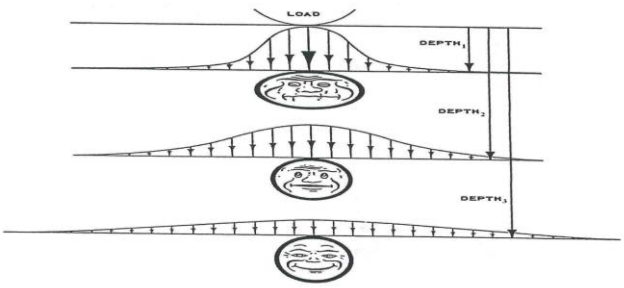

Fig 1. Effect of Traffic Load with Infill Depth

d)Effect of Ditch Width:

Ditches must be carefully backfilled in order to regain the stability originally present in the soil. Even in pipe

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installations where fairly good replacement of the material is achieved, voids or zones of poor compaction may be left under the haunches of the pipe, in the corrugations of metal pipe, or in bell holes of bell-and-spigot pipe. This can result in loss of vertical support within the ditch adjacent to the pipe. When this occurs, the weight of the backfill, which should be carried by material in the ditch beside the pipe, must be carried jointly by the pipe and the undisturbed soil beside the ditch. The narrower the ditch in which the pipe is laid, the less extra weight the pipe will receive.

e) Effect of Bedding

The load delivered to a pipe from above, whether it be the weight of the overlying soil or some surface load, must in turn be delivered by the pipe to the underlying soil. If firm support of the pipe by underlying soil is established only over a narrow width, as with a round pipe in a flat-bottom trench, the intensity of the load (stress) beneath the pipe will be large and failure is more likely. Some means of establishing firm support of the pipe over a wider band will reduce the load intensity beneath the pipe and consequently the likelihood of failure. This same effect magnified several times can occur with bell-and-spigot pipe if it is placed without cut-outs for the bells. Rock foundations are particularly critical in regard to load concentration, and an earth cushion must be provided to promote distribution of the load between the bottom of the pipe and the bedrock.

f) Effects of Improper Compaction on Pavement over Pipe:

It is possible for flexible pipe to be appreciably deflected and still carry most of its design flow. It is also possible for rigid pipe to accept the overloading which results from inadequate compaction of the side fill, in many cases without failure, and nearly always without failure to such a degree as to significantly decrease its flow capacity. But these conditions cannot be tolerated in overlying pavements, because the settlements that result cause pavement failures.

Greater settlement of backfill adjacent, rather than over, rigid pipe after construction of the overlying pavement leads to loss of pavement support on either side of the pipe with resultant pavement failure. Although two types of action leading to failure are involved here, the preventive measures are the same for both. The Inspector must see that good compaction is accomplished beside and over the pipe. In each case, this means more uniform pavement support.

g) Transmission of Load or Effect of Flexibility of Pipe on Supporting Ability:

Loads applied to a soil mass are transmitted downward through it along regular, smoothly flowing paths or lines. Loads applied to small areas are transmitted downward and outward from the centre of the load with intensities decreasing as the spreading increases. Broad loads, such as embankments, applied over wide areas are transmitted vertically downward along generally parallel paths with slowly diminishing intensities. When it is necessary to place an object such as a pipe in otherwise continuous soil, it will receive what might be considered as its proper share of the load only if it does not significantly change the pattern of load distribution within the soil medium. A pipe which is more rigid than the surrounding soil will stiffly accept more than its fair share of the load and cause the soil beside the pipe to be less heavily loaded. A pipe able to compress more than the surrounding soil will yield, or "shed", some of the superimposed load to the soil beside it.

h) Pipe Reaction to Load for Rigid Pipe:

Pipe of this type is ordinarily stiffer than the soil in which it is embedded. Thus the tendency is for the pipe to be deflected vertically less than the adjacent soil. This, in severe cases, leads to a hump over the pipe or low places on either side of it, and also results in the pipe carrying more than its proper share of the load from above. Here again, good compaction beside the pipe during installation will minimize the effect. The Designer ordinarily assumes that the soil surrounding the pipe has a density and therefore stiffness at least as great as the undisturbed adjacent soil. The Inspector must insure that adequate density is obtained otherwise failures will result.

Different Pipe Installation Conditions:

a) Trench Condition

Trench installation of conduit is most preferred from the standpoint of structural advantage and long term operational costs. In order to establish trench conditions, the minimum trench shapes must conform to the diagrams shown in Figure

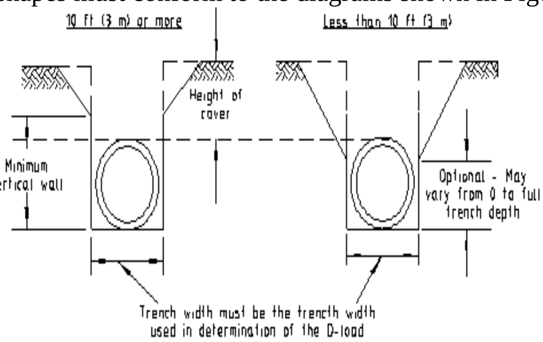

Fig. 2 Permissible Trench Shapes

Pipe has four basic installation conditions, as shown in figure.

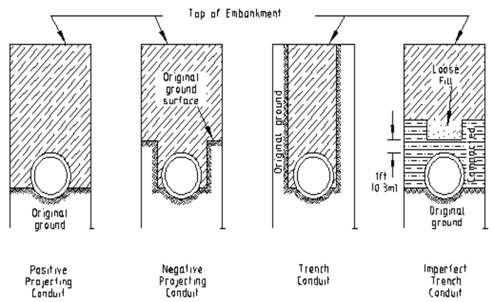

Fig. 3 Pipe Installation Conditions

b) Positive Projecting (Embankment)

Positive projecting installation, sometimes termed "embankment installation," is the simplest technique and has the most economical first cost. However, operationally, it does not serve to relieve any structural loading from above the conduit and may result in failure or high maintenance costs during the life of the structure.

c) Negative Projecting (Embankment)

Negative projecting conditions are more costly than the positive projecting conditions. Negative projection provides some loading relief from the conduit due to the frictional interface between the trench boundaries and the backfill. See fig. for a schematic of this effect. Negative projection conditions normally become cost-effective only when fill heights approach 30 ft. (10 m).

d) Imperfect Trench

The imperfect trench condition is usually more costly than any of the other three installation conditions shown. As with negative projection installation, imperfect trench installation normally becomes cost-effective only when fill heights approach 30 ft. (10 m).

Classes of Beddings

a) Class A Bedding

1) For Earth Foundation

In this method, the bottom of the pipe is bedded in plain or reinforced concrete of suitable thickness. The pipe is evenly supported on a continuous concrete cradle of monolithic cross-section if unreinforced. The width of the cradle is not less than the external diameter of the pipe plus 200 mm. The thickness of the cradle under the pipe is not less than one-quarter of the internal diameter of the pipe and the cradle extends up the barrel of the pipe for a vertical distance equal to X times the external diameter of the pipe, where X = 1/4 to 1/6. The compressive strength of the concrete in the cradle shall be not less than $15N/mm^2$ at 28 days. Selected fill material, free from clay lumps retained on a 75-mm sieve and from stones retained on a 26.5 mm sieve, is placed around and over the pipe and compacted in layers not exceeding ISO mm thick to a consolidated height of 300 nun above the top of the pipe.

2) For Rock Foundation

The pipe is evenly supported on a continuous concrete cradle, of monolithic cross section if unreinforced. The thickness of the cradle under the pipe is sufficient to allow adequate compaction of the concrete, but in no case it shall be less than twice the nominal size of the coarse aggregate or 50 mm whichever is the greater. The cradle extends up the barrel of the pipe for the vertical height XD assumed in the design. The compressive strength of the concrete in the cradle shall be not less than 15 N/mm' at 28 days. Selected fill material, free from clay lumps retained on a 75 mm sieve and from stones retained on a 26.5 mm sieve, is placed around and over the pipe and compacted in layers not exceeding 150 mm thick to a consolidated height of 300 mm above the top of the pipe.

b) Class B Bedding

1) For Earth Foundation

The pipe is evenly bedded on a continuous cushion of compacted sand or earth. The thickness of the cushion under the pipe is not Jess than 75 mm. The foundation is shaped concentrically with the pipe for a width not less than 0·6 times the external diameter of the pipe. FilJ material, free from clay lumps retained on a 75-mm sieve and from stone retained on a 26.5 mm sieve, is placed around and over the pipe and compacted in layers not exceeding 150 mm thick to a consolidated height of 300 mm above the top of the pipe.

If the fill material at the sides of the pipe and to a height of 300 mm above the top of the pipe is compacted to the same density as that of the foundation material, or to 90 percent of the maximum density at optimum moisture content as determined by a suitable method of test, a load factor of 2-5 shall be used; and If the fill material at the sides of the pipe is compacted to a lesser density than specified in (a) above, a load factor less than 2·5 depending on the density achieved, should be used. The minimum load factor that shall be used is 1.9.

2) For Rock Foundation

The pipe is evenly bedded on a continuous cushion of compacted sand or earth. The thickness of the cushion under the pipe is not less than 40 mm for each meter height of the fill material over the top of the pipe or 200 mm whichever is the greater. Care is taken to ensure that the pipe is not supported solely on the socket, if any, for example, a chase is excavated in the foundation material to prevent the socket from bearing on the foundation. The cushion extends up the barrel of the pipe for a vertical height of not less than one-quarter of the external diameter of the pipe. The width of the cushion is not less than the external diameter of the pipe plus 200 mm. Selected fill material, free from clay lumps retained on a 75mm sieve and from stones retained on a 26.5 mm sieve, is placed around and over the pipe and compacted in layers Not exceeding ISO nun thick to a consolidated height of 300 mm above the top of the pipe.

If the fill material at the sides of the pipe and to a height of 300 mm above the top of the pipe is compacted to the same density as that of the foundation material or to 90 percent of the maximum density at optimum moisture content as determined by a suitable method of test, a load factor of 2.5 shall be used. If the fill material at the sides of the pipe is compacted to a lesser density, a load factor less than 2.5, depending on the density achieved, should be used, The minimum load factor that shall be used is 1·9.

c) Type C Bedding

1) For Earth Foundation

The pipe is evenly supported on an earth foundation shaped to fit the barrel of the pipe for a width not less than one-half of the external diameter of the pipe. Fill material, free from clay lumps retained on a 75 mm sieve and from stones retained on a 26.5 mm sieve, is placed around and over the pipe and compacted in layers not exceeding 150 mm thick, to a consolidated height of 150 mm above the top of the pipe.

If the fill material at the sides of the pipe and to a height of ISO mm above the top of the pipe is compacted to the same density as that of the foundation material, or to 90 percent of the maximum density at optimum moisture content as determined by a suitable method of test, a load factor of 1·9 shall be used. If the fill material at the sides of the pipe is compacted to a lesser density a load factor less than 1.9 depending on the density achieved should be used. The minimum load factor that shall be used is 1.5.

2) For Rock Foundation

The pipe is evenly bedded on a continuous cushion of compact sand or earth. The thickness of the cushion under the pipe is not less than 20 rom for each meter height of fill material above the top of the pipe or 150 min, whichever is the greater. The cushion extends up the barrel of the pipe for a vertical height for not less than one-fifth of the external diameter of the pipe. The width of the cushion is not less than the external diameter of the pipe plus 200 mm. Fill material, free from clay lumps retained on a 75 mm sieve and from stones retained on a 26.5 mm sieve, is placed around and over the pipe and compacted in layers not exceeding 150 mm thick to a consolidated height of 150 mm above the top of the pipe.

If the fill material at the sides of the pipe and to a height of 150 mm above the top of the pipe is compacted to the same density as that of the foundation material or to 90 percent of the maximum density at optimum moisture content as determined by a suitable method of test, a load factor of 1·9 shall be used. If the fill material at the sides of the pipe is compacted to a lesser density, a load factor less than 1·9, depending on the density achieved, should be used. The minimum load factor that shall be used is 1.5.

d) Type D Bedding

1) For Earth Foundation

The pipe is laid on a foundation which does not fit the barrel of the pipe, but if the pipe is socketed a chase is made in the foundation to prevent the socket from bearing on the foundation. No special attempt is made to select and compact the fill material. The use of this method is not recommended. Load factor Ft shall be 1·1.

2) For Rock Foundation

The pipe is bedded on a continuous cushion of earth not less than 100 mm thick, on a foundation shaped approximately concentric with the barrel of the pipe. The cushion extends up the barrel of the pipe for a vertical height not less than one-tenth of the external diameter of the pipe. No special attempt is made to select and compact the fill material. The use of this method is not recommended. A load factor of 1·1 shall be used.

Various Loads Acting on Pipe and Load cases

a) Internal water Pressure or Static Pressure (Ps):

$$\mathbf{P_s = 9.81\left(\frac{3\pi}{4}\right)\left(\frac{d^4}{4}\right)}$$

Where, $\mathbf{P_s}$ -equivalent water load in KN/m^2

d- Internal diameter of the pipe in m

In general, the water load is not significant for small pipes of less than 600 mm diameter. The equivalent water loads on pipes of 600 mm to 1800 mm diameter are as below:

Table No. 1. Equivalent water loads.

Diameter in mm	Equivalent water load in KN/m
600	2.1
750	3.3
900	4.7
1050	6.4
1200	8.3
1350	10.6
1500	13.0
1650	15.8
1800	18.8

b)Internal Water Hammer Pressure

$$\mathbf{P_{h,max} = \frac{14.6}{\sqrt{1+\frac{Kd}{t}}} \times V}$$

Where, $\mathbf{K = \frac{E_w}{E_p}}$

c) Total Internal Pressure or Hydrostatic Pressure

The maximum internal pressure likely to come under worst circumstances is usually taken equal to sum of full static pressure and water hammer pressure.

$$\textbf{Total maximum internal pressure} = \textbf{Static Pressure} + \textbf{Water Hammer Pressure}$$

$$\mathbf{P = P_s + P_h}$$

Due to this hoop stress and longitudinal stresses will develop.

Hoop stress will be, $\boldsymbol{\sigma} = \mathbf{\frac{pd}{2t}}$

Longitudinal stress will be, $\boldsymbol{\sigma} = \mathbf{\frac{pd}{4t}}$

d) External Load Due to Weight of fill material as per IS 783- 1983:

In Positive projection embankment condition. The pipe is laid in a shallow excavation with its top projecting above the adjacent undisturbed foundation material. The vertical load transmitted to the pipe is usually greater than the load due to the weight of the fill material above the top of the pipe because settlement of the fill material adjacent to the pipe transfers additional load to the pipe by friction. It is advantage therefore, to compact the fill material adjacent to the pipe to maximum density.

$$\mathbf{W_e = C_e \gamma B^2}$$

Where value of C_e is given in IS 783- 1985 from Fig. 1

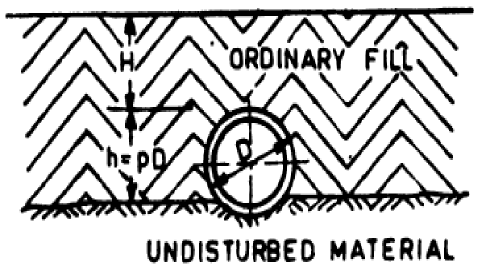

Fig 4. Pipe Laid Under Positive Projection Embankment Condition

e) Vertical load on a pipe due to superimposed concentrated load or Wheel Load:

The vertical load on a pipe due to a superimposed concentrated load *P* shall be calculated from the following formula:

$$\mathbf{We = Cp \times \frac{P \times \propto}{l}}$$

Where, C_p is given in figure 3 of IS 873- 1985 appropriate to the ratios of $\mathbf{\frac{l}{2H}}$ and $\mathbf{\frac{B}{2H}}$

l-The length of the pipe assumed to be carrying the concentrated load.

It may be calculated from the following formula but should not exceed the length of the pipe:

$$\mathbf{l = 1.15H + 2D + S}$$

Where atleast 300 mm of consolidated earth or equivalent cover cannot be provided, the wheel loads shall be assumed to be applied directly to the pipe. Magnitude of such forces decreases with increase in height of fill.

Experimental Work

For the design, modelling and Comparisons purpose following problem is considered as:

1)Consider a reinforced concrete pipe laid under positive embankment condition.
2)Internal diameter of pipe (d)-700 mm
3)Wall thickness or pipe thickness (t)-50 mm
4)External diameter of pipe (D) = 700+2×50 = 800 mm
5)Width of trench (Assumed), B=800+300+300= 1400 mm
6)Unit weight of fill material (W) - 18 KN/m^3
7)Height of embankment fill over the top of pipe, H= 2 m
8)Bedding and foundation material – Positive embankment condition: Bedding Type A: Earth foundation
9)Velocity of water in pipe, V=3 m/s
10)Settlement ratio, r_s= +0.8 (for positive embankment condition)
11)Projection ratio, P= 0.75

Step 1: Calculation of Internal water pressure and hoop tension acting on pipeline

a)Internal water pressure or Static pressure:

$$\boldsymbol{P_s = 9.81\left(\frac{3\pi}{4}\right)\left(\frac{d^2}{4}\right)}$$

$$\boldsymbol{P_s = 9.81\left(\frac{3\pi}{4}\right)\left(\frac{0.7^2}{4}\right)}$$

$$\boldsymbol{P_s = 2.8315\ \mathbf{KN/m2}}$$

b)Water hammer pressure

$$\mathbf{P_{h,max} = \frac{14.6}{\sqrt{1+\frac{Kd}{t}}} \times V}$$

For concrete pipes K=0.1

$$\mathbf{P_{h,max} = \frac{14.762}{\sqrt{1+\frac{0.1\times 0.7}{0.05}}} \times 3}$$

$$\mathbf{P_{h,max} = 28.5865\ KN/m^2}$$

c) Total internal pressure acting on pipe

$$\textbf{Total internal pressure} = \textbf{Static load pressure} + \textbf{Water Hammer Pressure}$$

$$\mathbf{P = P_s + P_h}$$
$$\mathbf{P = 2.8315 + 28.5865}$$
$$\mathbf{P = 31.42\ KN/m^2}$$

d) Hoop stress:

$$\boldsymbol{\sigma} = \frac{\mathbf{31.42 \times 0.7}}{\mathbf{2 \times 0.05}}$$
$$\boldsymbol{\sigma} = \mathbf{219.94\ KN/m^2}$$

e) Longitudinal stress:

$$\boldsymbol{\sigma} = \frac{\mathbf{31.42 \times 0.7}}{\mathbf{4 \times 0.05}}$$
$$\boldsymbol{\sigma} = \mathbf{109.97\ KN/m^2}$$

Step 2: Vertical load on pipeline due to fill material

As per IS 783-1983 load due to fill for positive embankment condition is given by,

$$\mathbf{W_e = C_e \gamma D^2}$$

The value of C_e can be obtained from Fig. 1 of IS 783-1983, Page No. 9

For $\frac{H}{D} = \frac{2}{0.8} = \mathbf{2.5}$ and $r_s \times P = 0.8 \times 0.75 = 0.6$

$C_e = 4$ after interpolation

$$\mathbf{W_e = C_e \gamma D^2}$$
$$\mathbf{W_e = 4 \times 18 \times 0.8^2}$$
$$\mathbf{W_e = 46.08\ KN/m}$$

Step 3: Vertical load on pipe due to superimposed loads (Traffic Load)

As per IRC equivalent single Wheel load is 41 KN i.e.

P= 41 KN

S=0.3 m

$$\mathbf{We = Cp \times \frac{P \times \propto}{l}}$$

$\alpha = 1$ When load is Static

$$\mathbf{We = Cp \times \frac{P \times \propto}{l}}$$
$$\mathbf{l = 1.15H + 2D + s}$$
$$\mathbf{l = 1.15 \times 2 + 2 \times 0.8 + 0.3}$$
$$\mathbf{l = 4.2\ m\ > 3m}$$
$$\mathbf{l = 3m}$$

And from fig. 3 of IS 783-1983, for $\frac{l}{2H} = \frac{3}{2\times2} = \mathbf{0.75}$ and $\frac{D}{2H} = \frac{0.8}{2\times2} = \mathbf{0.2}$,

$$\mathbf{C_p = 0.2}$$

After interpolation

$$\mathbf{We = 0.2 \times \frac{41 \times 1}{3}}$$
$$\mathbf{We = 2.734\ KN/m}$$

Step 4: Horizontal side pressure load due to side support offered by compacted fill:

Let the angle of internal friction for soil is, Ø= 30º

$$\mathbf{K_a = \frac{1 - Sin\emptyset}{1 + Sin\emptyset}}$$
$$\mathbf{K_a = \frac{1 - Sin30}{1 + Sin30}}$$
$$\mathbf{K_a = \frac{1}{3}}$$

At Top level of pipe,

$$\textbf{Side Pressure} = \mathbf{K_a \gamma H}$$
$$\textbf{Side Pressure} = \mathbf{\frac{1}{3} \times 18 \times 2}$$
$$\textbf{Side Pressure} = \mathbf{12\ KN/}\boldsymbol{m}^\mathbf{2}$$

At Bottom level of pipe,

$$\textbf{Side Pressure} = \mathbf{K_a \gamma (H + D)}$$
$$\textbf{Side Pressure} = \mathbf{\frac{1}{3} \times 18 \times (2 + 0.8)}$$
$$\textbf{Side Pressure} = \mathbf{16.8\ KN/}\boldsymbol{m}^\mathbf{2}$$

Our calculated side pressure distribution is trapezoidal. But for calculation purpose, equivalent pressure distribution is assumed which is rectangular in nature i.e. UDL.

$$\textbf{Equivalent Side Pressure} = \mathbf{\frac{12+16.8}{2}\ KN/}\boldsymbol{m}^\mathbf{2}$$
$$\textbf{Equivalent Side Pressure} = \mathbf{14.4\ KN/}\boldsymbol{m}^\mathbf{2}$$

Which is acting on half perimeter on each side. Therefore,

$$\textbf{Effective perimeter} = \mathbf{\frac{\pi D}{2} = \frac{\pi \times 0.8}{2} = 1.2566\ m}$$
$$\textbf{Side Pressure per meter} = \mathbf{14.4 \times 1.2566 = 18.1KN/m}$$

Step 5: Uplift Pressure Intensity:

$$\mathbf{\sigma_p = \gamma (H + D)}$$
$$\mathbf{\sigma_p = 18(2 + 0.8)}$$
$$\mathbf{\sigma_p = 50.4\ KN/}\boldsymbol{m}^\mathbf{2}$$

Which is acting on half perimeter at bottom. Therefore,

$$\textbf{Effective perimeter} = \mathbf{\frac{\pi D}{2} = \frac{\pi \times 0.8}{2} = 1.2566\ m}$$
$$\textbf{Side Pressure per meter} = \mathbf{50.4 \times 1.2566 = 63.335\ KN/m}$$

Step 6: Selection of bedding

As already mentioned, we assume the Type A bedding: Earth foundation

For Type A bedding, projection factor (P) is 0.75

$$\mathbf{P = \frac{h}{D}}$$

Where, h- Distance from the top of the pipe down to undisturbed foundation level

$$\mathbf{h = P \times D}$$
$$\mathbf{h = 0.75 \times 0.8}$$
$$\mathbf{h = 0.6m}$$

Step 7: Calculation of Load factor (F_e)

For positive embankment condition and Type A bedding for earth foundation

From section B- 10.4 of IS 783-1983 for projection ratio P = 0.75

F_e =3.9 (After interpolation)

Step 8: Selection of minimum test load

Total Load acting pipeline
= Vertical Dead load due to fill
+ Vertical load due to superimposed loads

Here internal water load is not considered because we are analyzing the pipeline for critical load condition and the versed condition is when the internal loads are considered as zero.

$$\textbf{Minimum required strength or load} = \mathbf{\frac{48.08}{3.9}}$$
$$\textbf{Minimum required strength or load} = \mathbf{12.52\ KN/m}$$

Computational Modelling in STAAD:

For STAAD Pro modelling some of the above forces are converted into nodal forces.

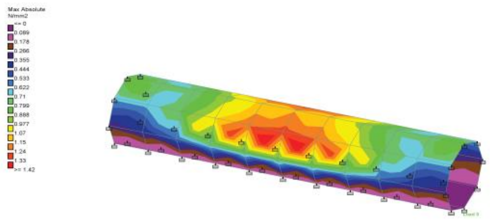

Fig 5. Plate contour stresses and STAAD Pro model

Table No. 2. Support Reaction Summery

	Node	L/C	Horizontal Fx (KN)	Vertical Fy (KN)	Horizontal Fz (KN)	Mx (KNm)	My (KNm)	Mz (KNm)
Max Fx	86	3	1.957	-6.304	-1.961	-0.004	0.048	0.042
Min Fx	84	3	-1.957	-6.304	-1.961	-0.004	-0.048	-0.042
Max Fy	82	3	1.116	9.655	4.574	0.145	-0.183	0.040
Min Fy	85	3	0.000	-7.676	0.000	0.000	0.000	0.000
Max Fz	81	3	0.000	9.141	14.716	0.053	-0.000	-0.000
Min Fz	1	3	0.000	9.141	-14.716	-0.053	0.000	0.000
Max Mx	82	3	1.116	9.655	4.574	0.145	-0.183	0.040
Min Mx	2	3	1.116	9.655	-4.574	-0.145	0.183	0.040
Max My	88	3	-1.116	9.655	4.574	0.145	0.183	-0.040
Min My	82	3	1.116	9.655	4.574	0.145	-0.183	0.040
Max Mz	46	3	-1.637	-0.900	-0.000	0.000	0.000	0.610

Table No. 3. Node Displacement summary

	Node	L/C	Resultant	Rotational		
			(mm)	rX (rad)	rY (rad)	rZ (rad)
Max X	47	3	0.000	0.086	-0.000	-0.000
Min X	43	3	0.000	0.086	-0.000	0.000
Max Y	1	3	0.000	0.000	0.000	0.000
Min Y	41	3	-0.000	0.130	0.000	0.000
Max Z	17	3	0.012	0.084	0.000	0.000
Min Z	65	3	-0.012	0.084	-0.000	0.000
Max rX	9	3	0.010	0.046	0.000	0.000
Min rX	73	3	-0.010	0.046	-0.000	0.000
Max rY	75	3	0.006	0.031	-0.000	0.000
Min rY	79	3	0.006	0.031	-0.000	-0.000
Max rZ	48	3	-0.000	0.066	0.000	-0.000
Min rZ	42	3	-0.000	0.066	0.000	0.000
Max Rst	41	3	-0.000	0.130	0.000	0.000

Table No. 4. Plate Centre Stress Summery

	Plate	L/C	Shear		Bending		
			SQX	SQY	Mx	My	Mxy
Max Qx	46	3	0.267	-0.000	-0.243	-0.033	0.004
Min Qx	43	3	-0.267	-0.000	-0.243	-0.033	-0.004
Max Qy	73	3	-0.008	0.077	-0.061	0.089	0.044
Min Qy	1	3	-0.008	-0.077	-0.061	0.089	-0.044
Max Sx	1	3	-0.008	-0.077	-0.061	0.089	-0.044
Min Sx	42	3	0.113	-0.003	0.774	0.140	0.003
Max Sy	73	3	-0.008	0.077	-0.061	0.089	0.044
Min Sy	33	3	0.085	-0.001	-0.532	-0.111	-0.007
Max Sxy	79	3	-0.022	-0.029	0.105	-0.023	-0.027
Min Sxy	74	3	0.022	-0.029	0.105	-0.023	0.027
Max Mx	34	3	0.113	0.003	0.774	0.140	-0.003
Min Mx	33	3	0.085	-0.001	-0.532	-0.111	-0.007
Max My	34	3	0.113	0.003	0.774	0.140	-0.003
Min My	41	3	0.085	0.001	-0.532	-0.111	0.007
Max Mxy	65	3	0.033	0.014	-0.257	-0.066	0.066
Min Mxy	72	3	-0.033	0.014	-0.257	-0.066	-0.066

Result and Conclusion

From the above study it is found that depth of fill governs the most of all factors. Greater the H/D value greater is the infill weight. Also node displacement summary gives maximum displacement values for load combination 3. Top nodes shows the maximum displacement in vertical direction while bottom nodes are free from node displacements.

Plate center and plate corner stresses are maximum for top plates, intermediate for middle side plates and minimum for bottom plates. End nodes shows zero rotational displacement and translational displacement in all direction.

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