International Journal of Engineering & Technology, *7 (3.36) (2018) 190-195*

International Journal of Engineering & Technology

Website: www.sciencepubco.com/index.php/IJET

Research paper

A Case Study on Design of Ring Footing for Oil Storage Steel Tank.

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Abstract

Foundations of large steel oil storage tanks considered as case study to conduct the correct steps of the foundation design. This study aims at understanding the principles of the geotechnical and structural design of foundation for heavy oil storage steel tank. The storage tank is used in the Iraqi oil industry at Basra, south of Iraq. The steel tank 26.78 m in diameter and 10 m height and the transmitted load is in pattern of a line load along the circumference of tank. A finite element model using SAP program was using to simulate the foundation as a ring footing. Two series of long driven piles were used along the circumference of tank to support the applied load from the tank and oil liquid and to be as a base of the ring beam reinforced concrete beam.

Keywords: *Oil storage tank, driven piles, ring footing, reinforced concrete ring beam, piles cap.*

1. Introduction

Cylindrical storage tanks form a mainly part of petroleum refineries, chemical plants and many other manufacturing units. They store inside them large volumes of hazardous products. Failure of such tanks can lead to severe environmental damage, loss of human life and big financial losses. Scientific literature suggests that differential settlement has been a major cause of destroy in such tanks. Therefore, the reliable estimation of loads constitutes an important step in design of foundations of oil tanks. The estimations vary quite significantly depending on the method adopted for settlement and bearing capacity calculations.

The present article deals with one such exercise carried out as reference to foundations of oil storage tanks constructed in AL-BASRAH area (south of Iraq).

The objective of the present study is to improve the foundation design of oil storage tank. The problems are designed conventionally depending on same ideas from Russian company (PeterGib) as shown in Fig. 1. The problem was simulated through three-dimensional finite element using SAP2000 software and compared with Meyerhof analytical equations.

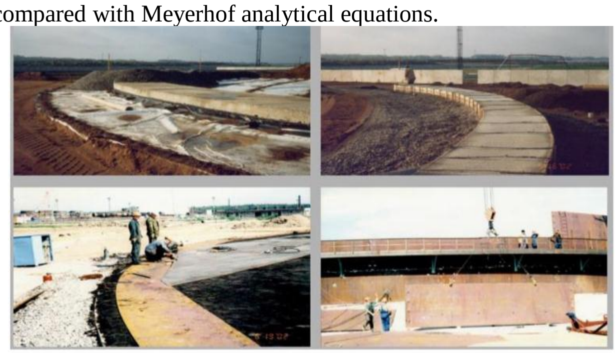

Fig1: Ring Shallow Foundation For Large Storage Tank Designed By Russian Company (Petergib), From Achieve Of The First Author Of This Article.

2. Details of Storage Tank and Site Investigation.

The oil tank is consisting of steel plates and reinforced using steel sections to increase stiffness of the tank. The dimensions of steel tank are 26.78 m in diameter and 10 m height as shown in Fig. 2. The tank is supported on the external edge along the circumference of tank and the transmitted load is as a line load along the circumference of tank. The calculated weight of empty steel tank was 200 ton (2000 kN).

A detailed soil investigation was planned and executed to map the soil strata. The investigations consisted of three boreholes and many standard penetrate test a. The boreholes were made up to a depth of 22.0 m. In each of the boreholes, standard penetration test (SPT) was conducted at 1.5m depth intervals. Representative sample collected through the SPT sampler were used for classification tests. Undisturbed samples collected in clay layers through thin walled samplers were used for shear and consolidation tests. Based on the field and laboratory test data, bore logs were prepared.It was observed that the subsoil conditions at the site are more or less identical at all bore hole locations. Accordingly, an average representative soil investigation including shear strengthparameters and SPT blows number for the site are summarized in table 1.

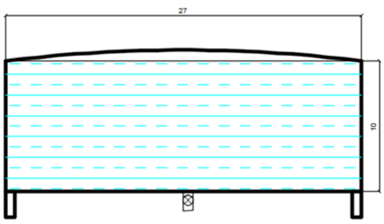

Fig 2: Section of Suggested Tank

It can be seen that the subsoil consists alternating layers of non-plastic silt sand of varying thickness up to 22.0m, the maximum depth of exploration.
The soil investigation report refers to weak soil with barring capacity 3 ton/m^2 for upper layer and depth 6 m.

3. Proportioning of foundation.

The oil tank foundations are generally proportioned based on conventional method. Meyerhof [7] concluded that the point bearing capacity, of a pile in sand generally increases with the depth of embedment in the bearing stratum and reaches a maximum value at an embedment ratio of $L_b/D = (L_b/D)_{cr}$.
Where:
L_b = The actual embedment length of the pile. *L.*
D = The diameter of the pile.
Euro code 7 describes three procedures for obtaining the characteristic compressive resistance $R_{c,k}$of a pile [3]:
(a) Directly from static pile load tests with coefficient ξ_1 and ξ_2 for *n* pile load
(b) By calculation from profiles of ground test results or by calculation from ground parameters with certain coefficients.

Table 1: Shear strength parameters and SPT blows number.

	B.H 1		B.H 2		B.H 3			
depth (m)	*SPT*	*Ø*	*SPT*	*Ø*	*SPT*	*Ø*	*Average SPT*	*Average Ø*
1	50	41				39	50	40
2		35	32	41	48	42	40	39
3	3		4		9		5	
4			3				3	
5	3		2		16	38	3	38
6			3			39		39
7								
8	3		3		34	33	3	33
9								
10		19.2	9		50	41	9	30
11								
12	27	39	32	30			30	34
13					50	39	50	39
14	50	28.3					50	28
15			50	27	41	24	45	25
16								
17								
18			45	19	48		46	19
19	50	38					50	38
20								
21	50	39	50	38	50	45	50	41
22								
average							**46**	**34**

Ø: is the angle of internal friction in degrees

(c) Directly from dynamic pile load tests with certain coefficient.
In the case of procedures (a) and (b) Euro code 7 provides correlation factors to convert the measured pile resistances or pile resistances calculated from profiles of test results into characteristic resistances.
The Architectural Institute of Japan [10] developed an ultimate bearing capacity formula which considers the size effect factor and now is widely used in Japan. It was developed by semi-experiments. The ultimate bearing capacity formula is expressed as follows [10]:

$$q_u = i_c \alpha c N_c + i_\gamma \gamma \beta B \eta N_\gamma + i_q \gamma D_f N_f \quad (1)$$

Where:
c: cohesion, γ: unit weight of soil, D_f: depth of embedment, *B*: footing width; N_c, N_q, N_γ: bearing capacity factors; i_c, i_q, i_γ: inclination factors, α and βis express the shape coefficient and α = 1 and β = 0.5, q_u: is ultimate vertical bearing capacity per unit area of footing (kN/m^2) and η: the size effect factor is defined as:

$$\eta = \left(\frac{B}{B_o}\right)^m \quad (2)$$

Where:
B_0: reference value in footing width *m*: coefficient determined from the experiment (*m* = -1/3 is recommended in practice).
The ultimate load-carrying capacity (Q_u) of a pile is given by the equation:

$$Q_u = Q_p + Qs \quad (3)$$

Where:
Q_p = load-carrying capacity of the pile point.
Qs = frictional resistance (skin friction) derived from the soil–pile interface.
Figure 3 illustrates the components of pile point and frictional resistance forces.
Many researchers published studies cover the estimation of the values of Q_p and Q_s. Famous investigations have been provided by Vesic[11], Meyerhof [7] and Coyle and Castello [4].
These studies adopted methods to determine the ultimate pile capacity.
The Terzaghi's bearing capacity equation of the unit base resistance at the base of the pile can be calculated as following:

$$q_b = (\sigma'_v)_b N_q + c'_b N_c \quad (4)$$

Where:
A_p = The cross section of the pile.
q = The overburden pressure at the tip of the pile.
N_q = The bearing factor.
c_b'= Cohesion of the soil under the base of the pile,
$N_c = (Nq - 1) \cot\varphi$.
For piles in sand, the point bearing capacity equal to:

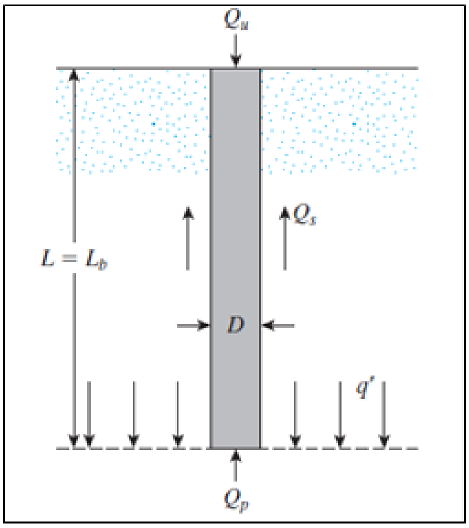

Fig 3: Ultimate load-carrying capacity of pile[5]

$$Q_p = A_p q_p = A_p N_q^* \quad (5)$$

Meyerhof [7] suggested a variation of N_q with soil friction angle Ø as shown in Fig. 4.
Janbu[6] presented equations to estimate capacity coefficients *Nq* for various soils

$$N_q = (\tan\phi + \sqrt{1+\tan^2\phi})^2 \exp(2\eta \tan\phi) \quad (6)$$

Where:
η is an angle defining the shape of the shear surface around the tip of a pile as shown in Fig. 5. The angle η ranges from π/3 for soft clays to 0.58π for dense sands.
The adopted values of bearing capacity factor *Nq* according to NAVFAC DM 7.2 [9] can be summarized in table 2.

Table 2: Friction angle φ vs. Nq [9]

ϕ' [°]	26	28	30	31	32	33	34	35	36	37	38	39	40
N_q for driven piles	10	15	21	24	29	35	42	50	62	77	86	120	145
N_q for bored piles	5	8	10	12	14	17	21	25	30	38	43	60	72

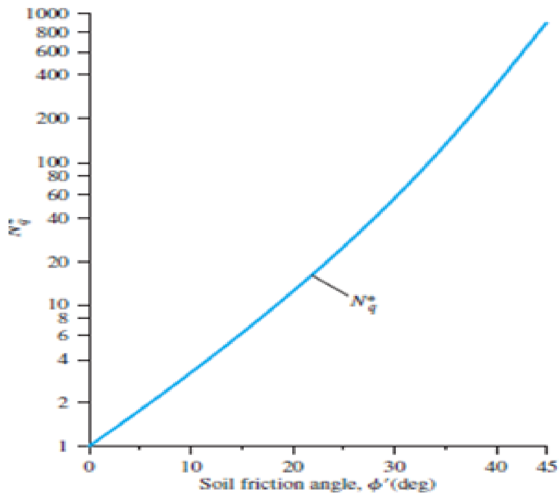

Fig 4: Variation of the maximum values of Nq with soil friction angle [7].

The frictional, or skin, resistance of a pile may be written as:

$$Q_s = \sum \rho \, \Delta L \; f \tag{7}$$

Where:
p = perimeter of the pile section.
ΔL = incremental pile length over which *p* and *f* are taken to be constant.

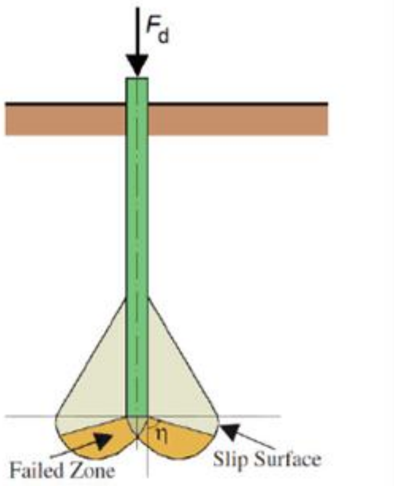

Fig 5: Shear surface around the base of a pile: definition of the angle η [6].

f = unite friction resistance at any depth z.
Also the main principles of pile capacity calculations presented in Euro code 7 and other methods can be explained.
The β-method is famous method to estimate the skin friction of piles, and the independent equation as following:

$$q_s(z) = \mu(z)k(z)\sigma_v'(z) = \beta(z)\sigma_v'(z) \tag{8}$$

McClelland, 1974 estimated β according to the following propositions:
β = 0.15 to 0.35 for compression and β = 0.10 to 0.25 for tension (for uplift pile).
Kraft and Lyons, 1990 suggested β according to the following propositions:
β = C tan(φ – 5), C = 0.7 for compression, C = 0.5 for tension (uplift piles)
Regarding to NAVFAC DM 7.2 [9], β =μ (z) K(z) = tanδ(z)K(z), Tables 3 and 4 indicate the varientsβand K(z):

Table 3: Pile skin friction angle (δ) [9]

Pile type	Pile-soil interface friction angle (δ)
Steel piles	20°
Timber piles	3/4 ϕ'
Concrete piles	3/4 ϕ'

Table 4: Lateral earth pressure coefficient (K).

Pile type	*K* (piles under compression)	*K* (piles under tension)
Driven H-piles	0.5–1.0	0.3–0.5
Driven displacement piles (round and square)	1.0–1.5	0.6–1.0
Driven displacement tapered piles	1.5–2.0	1.0–1.3
Driven jetted piles	0.4–0.9	0.3–0.6
Bored piles (less than 60 cm in diameter)	0.7	0.4

4. The Results and Discussion

The main aim of this study is to find out the optimum and economic footing for heavy storage tank when the diameter of the tank is large. The behavior of the deep foundation was analyzed using structural software program and the conventional equations. The suggested footing is as ring pile cap with diameter equal to the diameter of the oil tank. The pile will be distrusted along the circumference of the ring footing.

4.1.The Allowable Load on Pile.

The conventional equations used to estimate the allowable load on the individual pile was adopted. From table 1 the soil is sand compositions, so the Meyerhof equations to estimate the ultimate bearing capacity of soil were used.
After the total ultimate load-carrying capacity of a pile has been determined by summing the point bearing capacity and the frictional (or skin) resistance, a reasonable factor of safety should be used to obtain the total allowable load for each pile as [2]:

$$Q_{all} = \frac{Q_u}{FS} \tag{9}$$

Where:
Q_{all} = allowable load-carrying capacity for each pile.
FS = factor of safety.
In present study, the ultimate bearing capacity of the individual piles was estimated depending on the direct shear test and standards penetration test and the average value was selected.
Some of researcher's equations and procedure calculation of the allowable load on Pile are adopted in this study. The calculation was divided into two parts; the first related to unite base resistance and the second includes frictional resistance.

4.1.1 UNITE BASE RESISTANCE.

A- Meyerhof equations.

Q_b for sand =q¯. N_q. A_b
Using driven piles 0.35×0.35 m and L= 22 m
ɣ=15.5 kN/m^3 and W.T = 2m

q=15.5×2+5.5×20= 142 kN/m^2

N_q=50 for Ø=34 from figure 4

A= 0.35×0.35=0.1225 m^2

Q_b = 142 × 50 × 0.1225= 869.75 kN

B- Standard penetration test using Meyerhof equations.
L_b= 1.5m N= 50

Q_b = (40N) (Lb /B) Ab = 40 × 50 (1.5/0.35) × 0.1225= 1050 kN
Q $_{allowable}$ = 1563.375 /3= 521 use 500 kN.
C- Janbu Equation [6]
Q_b for sand =q¯. N_q. A_b
Using driven piles 0.35×0.35 m and L= 22 m
ɣ=15.5 kN/m^3 W.T = 2M

q=15.5×2+5.5×20= 142 kN/m^2

$$N_q = (\tan\phi + \sqrt{1+\tan^2\phi})^2 \exp(2\eta\tan\phi) \quad \textbf{[6]}$$

$$N_q = (\tan 34 + \sqrt{1+\tan^2 34})^2 \exp(2\times 0.58\pi\tan 34) = 41.32$$

A= 0.35×0.35=0.1225 m^2

Q_b = 142 × 41 × 0.1225= 713.2 kN

4.1.2 Frictional Resistance.

A- Meyerhof equations [7].
Q_s for Sand = K σ tan Ø As
k= 1-sin30 = 0.5

σ_0 =15.5 × 2 + 5.5 × 9 = 80.5 kN/m^2
As= 4 × 0.35 × 22= 30.8 m^2
Qs= 0.5 × 80.5 × tan (0.75 ×34) × 30.8= 591 kN

B- Standard penetration test using Meyerhof equations.
L_s= 11 m and N= 40
half-length neglected because is weak soil.

A_s= 4 × 0.35 × 11=15.4m^2

Q_S= (N) As = 40 × 15.4 =616 kN
C- McClelland Equations [8].
For Sand

$$Q_s = \beta\sigma_v A_s$$

β = 0.15 to 0.35 use 0.35
σ_0 =15.5 × 2 + 5.5 × 9 = 80.5 kN/m^2
As= 4 × 0.35 × 22= 30.8 m^2

Qs= 0.35 × 80.5 × 30.8= 867.79 kN
Table 5 explains the final results of the calculation for different researchers for the two parts of pile resistance loads. The results of conventional equations are seem to be compatible for most researcher equations.
So, the following values were considered in the design of pile foundation.
Q_{total}= 840.885 + 710.87 = 1551.755 kN.

$Q_{allowable}$ = 1551.755/3 = 517.25 kN for factor of safety = 3
Use the allowable load on pile = 500 kN.

Table 5: The final results of the calculation for different researchers.

No.	Researcher Name	Base Resistance (kN)	Skin Friction Resistance(kN)
1	Meyerhof	869.75	591
2	Meyerhof (SPT)	1050	616
3	Janbu and McClelland	713.2	867.79
4	NAVFAC DM	730.59	768.69
Average		840.885	710.87

4.2 Number of pile and spacing.

The applied load = 58327 kN

No. of piles = 58327/500= 116.6 so use 120 piles.

Use two rows of piles each row have 60 piles.
The longitudinal spacing between pile =

$$\frac{\pi \times D}{\text{No. of piles}} = \frac{\pi \times 26.78}{60} = 1.402m > 3d$$

The sectional spacing also will be taken = 1.4 m > 3 diameter of pile it is OK.

4.3 Shear and Structural Design of Piles Cap.

The structural design divided into part; the first part is shear check of cap thickness and the second part is the flexural design (ACI-Code 2011).

4.3.1 Shear check.

The width of pile cap = 1.4 +2.5 d = 2.3 m.
Assume the thickness of piles cap =1 m.
Weight of pile cap = 2.3 ×1 m× 24 kN/m3 = 55.2 kN/m < 30 kN/m^2 × 2.3= 69 kN/m
Vu (shear stress) = 58327 ×1.5 kN / π × 26.78m = (1040 + 69)/2 = 554.5 kN/m.
Beam with 0.3 × 0.4 will be added to carry the line load from the oil tank as shown in Fig. 5.

Table 6: ACI -wide-beam action (article 11.11.1.1, 2011)

ϕ = 0.85	SI	Fps	ACI Code Reference
Wide-beam	$\phi\sqrt{f'_c}/6$	$2\phi\sqrt{f'_c}$	Art. 11.3.1.1
Two-way action when $\beta \le 2$	$\left(1+\frac{2}{\beta}\right)\frac{\phi\sqrt{f'_c}}{6}$	$\left(2+\frac{4}{\beta}\right)\phi\sqrt{f'_c}$	Art. 11.12.2.1
but not more than:	$v_c = \frac{\phi\sqrt{f'_c}}{3}$ MPa	$4\phi\sqrt{f'_c}$ psi	
	$\beta = \frac{\text{Col. length}}{\text{Col. width}}$		
	f'_c = MPa	psi	

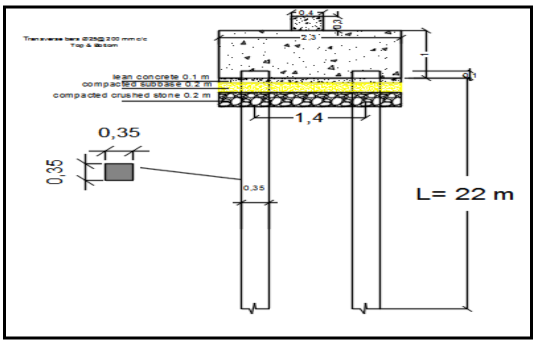

Figure 6: Section in pile cap.

Assume f_c= 30 Mpa and fy= 350 Mpa

$$V_c = 0.17 \times \phi \times \sqrt{f'_c} \times b \times d = 0.17 \times 0.85 \times \sqrt{30} \times 1 \times 0.9 \times 1000 = 712 \text{ kN/m}$$

$$V_c = 0.17 \times \phi \times \sqrt{f'_c} \times b \times d = 0.17 \times 0.85 \times \sqrt{30} \times 1 \times 0.9 \times 1000 = 712 \text{ kN/m}$$

4.3.2. Reinforcement Design.

In this part, the finite element method was used to represent the foundation model. To simulate the model, SAP 2000 software program was utilized. The sand soil and the piles were simulated as spring with constant stiffness and the pile cap as concrete plate. Stiffness of pile parameter was required to simulate the sand and piles. The modulus of elasticity, thickness, compressive strength and the unite weight were needed to simulate the piles cap.

Figures 7 and 8 represent the model which was analyzed using Sap 2000 software.

Weight of pile cap = 2.3 ×1 m× 24 kN/m^3 = 55.2 kN/m < 30 kN/m^2 × 2.3= 69 kN/m .

So, the weight from piles cap will be designed on the ground surface.

Stiffness of pile $K = \frac{EA}{L} = \frac{4700 \times \sqrt{30} \times 1000 \times 0.35^2}{22} = 143341.48$

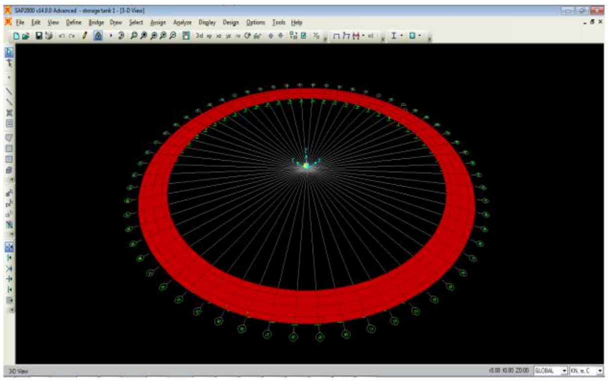

Fig 7: Finite Element Model.

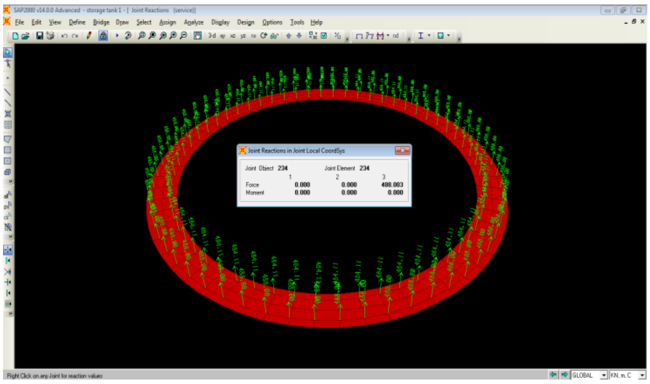

Fig 8: Reactions on piles.

From figure 8 the applied load on pile = 488 kN < 500 kN ok

Figures 9 and 10 illustrate the moment diagram for pile cap in x and y direction respectively.

Max. x- Direction moment = 369 kN.m

Assume d= 0.9 m, fy= 350 Mpa

$$A_s = = \frac{m}{j.d.fy} = \frac{369 \times 10^6}{0.87 \times 900 \times 350} = 1480 mm^2$$

Use Ø 25 at 250 mm give 1960 mm^2

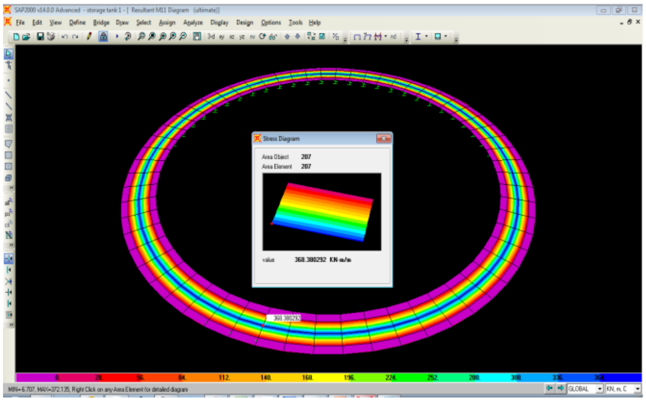

Fig 9: Moment in x-direction

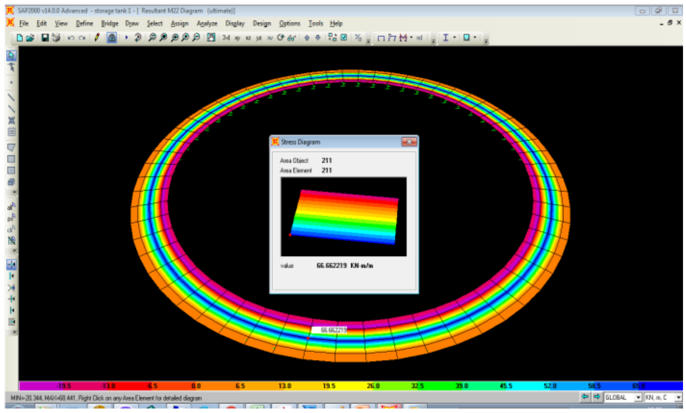

Fig 10: Moment in y-direction

Max. y- Direction moment = 67 kN.m

$$A_s = = \frac{m}{j.d.fy} = \frac{67 \times 10^6}{0.87 \times 900 \times 350} = 245 mm^2$$

CHECK OF NUMERICAL ANALYSES.

The numerical model of the present work has been checked by comparing some of the present results with that computed using the conventional method. The results of shear force and the reaction on the piles were considered as base of the comparison. Figure 11 shows the maximum shear analysis resulted from SAP program analysis.

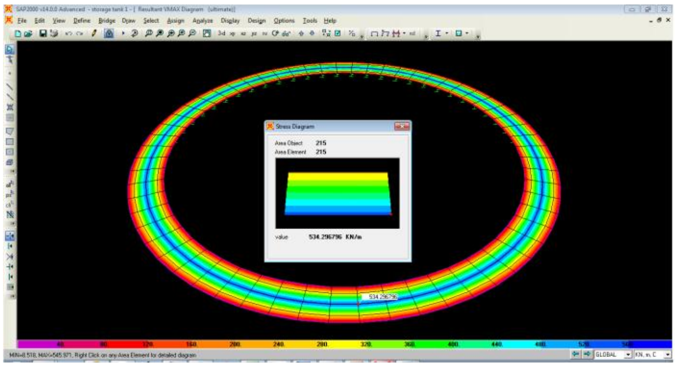

Fig 11: Maximum shear analyses.

Maximum shear force from finite element= 534.3 kN/m ≈ 554.5 computed so it is OK.

Applied load = 58327 kN

Weight of pile cap = 69 kN/m.

Ultimate applied load = ((58327 / π × 26.78) + 69) × 1.5 = 763 kN/m

The load on each pile of two side = 763/2 = 381 .5 kN ≈ 369 kN from SAP analyses.

The final design of the suggested footing is shown in Figs 12 and 13

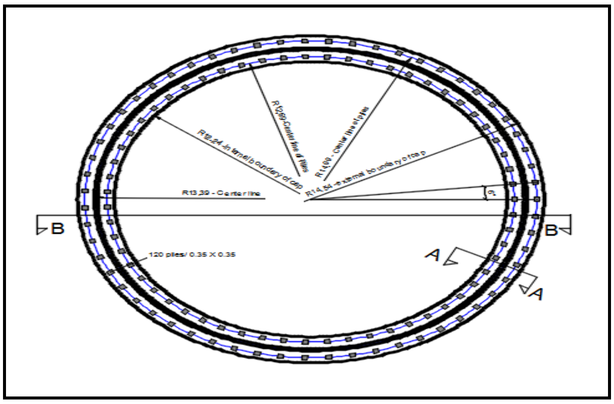

Fig 12: Plan of suggested ring footing.

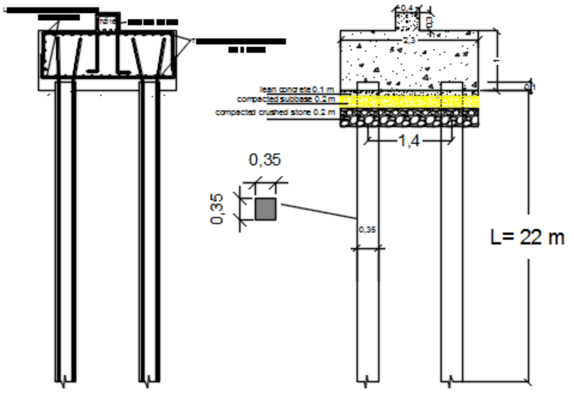

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Fig 13: Sections of the suggested footing.

5. Conclusions.

The numerical analysis was carried out to estimate the reaction, shear and moment forces on footing supports heavy oil tank. Also the design was implemented using the conventional method using Meyerhof [7], Janbu [6], McClelland [8] and NAVFAC DM 7.2 [9] equations. The results of conventional equations are seem to be compatible for most researcher equations. The present work can be drawn that the results show that using ring footing rested on piles is suitable and economic solution to support the heavy oil storage tank on weak soil. The results obtained from the SAP analyses showed generally agreement with Meyerhof analytical equation. The finite element analysis using structural software program give acceptable results of the reactions, shear and moment values.

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