Static equilibrium analysis for pipeline protection design in Iran LNG port

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ABSTRACT
The present paper deals with the protection design of intake seawater pipeline that will be constructed in the LNG Project. In this study by using of limit equilibrium approach, the geometry of the protection layer with the viewpoint of holding capacity of drag anchors in cohesionless soils is estimated. All technical and executive points regarding pipeline protection such as anchor's type, ship load, geotechnical properties of the protected zone and underlying seabed, and finally procedure to calculate geometry of protection layer are suitably addressed.

RÉSUMÉ
Le présent document concerne la conception de protection de l'oléoduc d'eau de mer d'admission qui sera construit dans le projet LNG. Dans cette étude, avec l'approche d'équilibre limite, la géométrie de la couche de protection avec le point de vue de la capacité de rétention des ancres glisser dans les sols pulvérulents est estimée. Tous les points techniques et d'exécution en matière de protection de pipelines, comme l'ancre de type de charge des navires, propriétés géotechniques de la zone protégée et sous-jacents des fonds marins, et enfin la procédure de calcul de la géométrie de la couche de protection sont convenablement pris en compte.

1 INTRODUCTION

Seawater pipelines that are located near to the harbor are susceptible to damage because of contacting with anchors. So, designing a safe protection is essential every time (Figure 1). But this fact should be considered that designing this type of protection because of the interaction among anchor, anchor chain, sea bed and rock armour is so complex. Usually for solving this complicated issue, model testing has been performed that is because of lacking a proper analytical solution. But generally, the protection design geometry depends on three factors that can be classified as soil, anchor, and chain types.

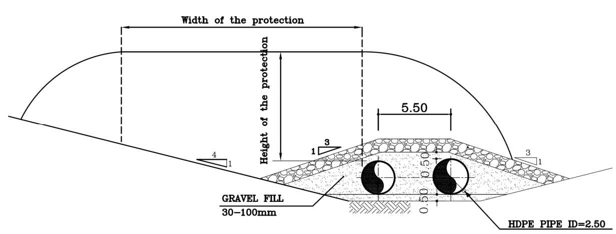

Figure 1. Typical view of pipeline protection

In LNG project of Iran is faced with such a problem and protection should be designed for intake seawater pipeline. Figure 2 illustrates the general layout and construction stages of the LNG port. Iran LNG Company has planned to build LNG plant on the south west coast of Iran, in Tombak. The LNG Plant is located at the west end of the area where three other LNG plants are planned to be constructed. The marine facilities of current project include Harbour, LNG Jetty, LPG Jetty, Seawater Offshore Intake and Outfall Structures, and Seawater Pipelines with the maximum length of 300m.

Whereas, using of model testing is so costly and timely, performing an analytical design by implementing the studies that have been done in this field is preferred. To achieve this purpose, anchor's type, ship load and geotechnical properties of the protected zone and underlying seabed should be determined.

The methodology of analysis that will be used in this study is based on the static equilibrium of drag anchors during embedment in cohesionless soil. Originally, a limit equilibrium method is developed to allow the anchor holding capacity to be predicted given a specific anchor geometry and position relative to the free surface. The method of Neubecker and Randolph (1996a) that was used in the current study is based on earlier approach of Le Lievre and Tabatabaee (1979, 1981), but is extended to include a three-dimensional soil wedge, which is consistent with experimental observation. The solution is also extended to be more applicable for anchors at a pre-ultimate stage by incorporating a force behind the anchor fluke. Moreover, in order to calibrate the method a series of centrifuge model anchor tests has been performed by Neubecker and Randolph (1996a).

By using of this information, all technical and executive points regarding pipeline protection such as length and width of protection are suitably addressed in four steps respectively:

- Determining geotechnical properties;

- Determining the type and applied load to the anchor;
- Analyzing and determining the applied force on the anchor;
- Determining height and width of protection; and
- Summery and conclusion.

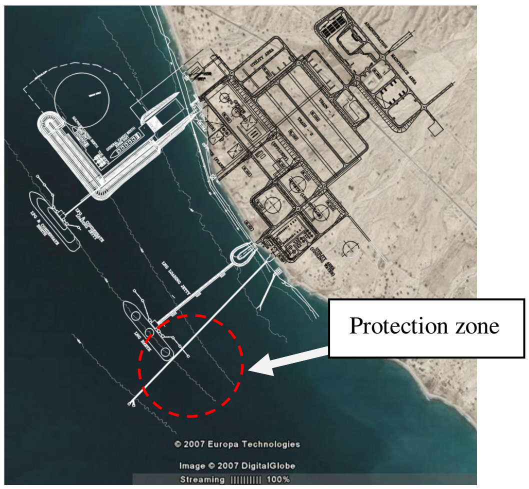

Figure 2. General layout of the LNG port

2 GEOTECHNICAL PARAMETERS OF THE PROTECTED ZONE AND UNDERLYING SEABED

The geotechnical parameters of the rock armour as well as seabed soil layers to be used in the analyses are presented herein. Site investigations were performed to recognize the geotechnical characteristics of seabed layers in the construction site. Based on the drilled geotechnical boreholes and subsequent surveys, two separate zones were recognized around protection site. Figure 3 shows the recommended zonation indicating that the protection area site in the zone B. In this zone, the upper layer having a maximum thickness of 13.0m is a silt layer with low plasticity (ML) and the lower thick layer is a dense composite soil (GC-GM). Geotechnical estimation of characteristics of this layer is shown in Table 1 based on the results of in-situ index tests including SPT, CPTU, and PMT and also laboratory tests (direct shear and triaxial) conducted on the undisturbed and reconstituted specimens.

Because of simplicity in construction, the geotechnical properties of the protection materials were selected the same as those used in the filter of LNG project's breakwater. In the Table 2, the proposed geotechnical parameters of the breakwater filter are presented. As it is seen, two types of parameters, namely "Type I" and "Type II", were recommended by the design criteria for the analyses. In the Type I, an apparent cohesion has been taken into account. Slope stability analyses of the breakwater demonstrated that material Type II, typically, provides more critical safety factors than material Type I. In the current report, therefore, only material Type II is implemented in the analyses.

3 DETERMINING THE TYPE AND APPLIED LOAD TO THE ANCHOR

Increasing demands on anchoring system is derived from developing floating system. There are several options for providing an anchorage, such as gravity-type anchors or pile anchors, but by far the most economical and hence preferable system is to adopt high capacity drag anchors (Neubecker and Randolph 2006).

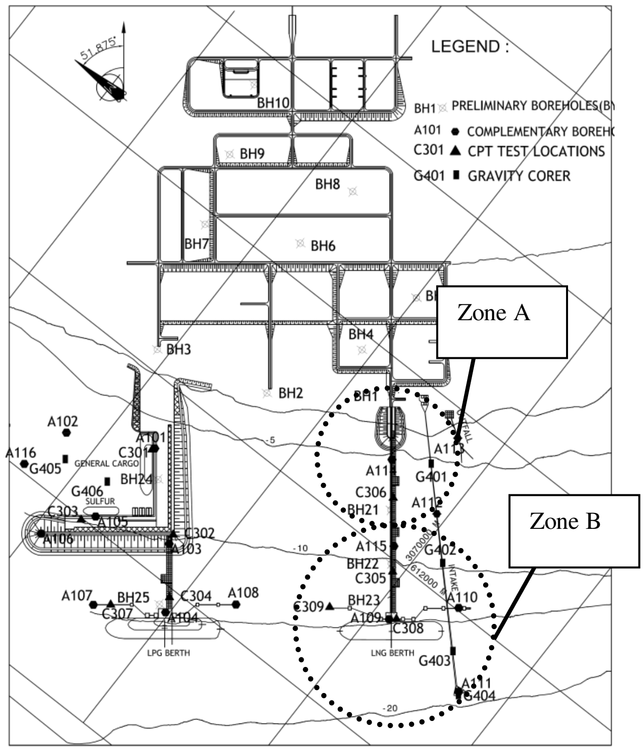

Figure 3. Geotechnical site investigation results, zones A and B

Table 1. Geotechnical parameters of the seabed soil layers (part II, zone B)

Parameters	Unit	Layer 1	Layer 2
Depth	m	0~13	>13
Classification	-	ML	GC-GM
Relative density	%	40	75
Fine content	%	70	29
Liquid limit	%	26	19
Plasticity index	%	NPI	5
Dry density	ton/m^3	1.47	2.03
Saturated density	ton/m^3	1.96	2.31
Specific gravity	-	2.7	2.71
Cohesion	ton/m^2	0	0
Integral friction angle	deg	28	39
Elastic modulus	ton/m^2	700	14000
Poisson's ratio	-	0.35	0.25

Drag embedment anchor and vertically loaded anchor are two common types of drag anchors. Drag embedment anchors have been designed to penetrate into soil as deeply as possible to develop the maximum capacities of the fluke and shank. As the anchor penetrates, the anchor capacity increases with depth due to the increase in failure wedge. Today drag embedded anchors are usually employed for temporary mooring systems with catenaries mooring in deep water. So, this type of anchor is chosen for doing analysis in this paper. Because of existence the different types of drag embedded anchors with complex geometry and for simplifying the analysis of applied load on the anchor some simplifications should be done on the complex geometry of anchor (Figure 4).

Table 2. Proposed geotechnical parameters for protection layer

Design parameters	Unit	Filter materials	
		Type I	Type II
Dry density[1]	kg/m^3	1700	1700
Saturated density[2]	kg/m^3	1900	1900
Integral friction angle[3]	degree	38	41
Cohesion[4]	kg/m^2	0.2	0
Elastic modulus[5]	kg/m^2	1000	1000
Poisson's ratio[6]	kg/m^2	0.3	0.3

[1,2]Based on results presented in "Eurocode 8"
[3,4]According to proposed values in "OCDI", "CUR", and "Neves (1991)"
[5,6]Based on parameter range collected in "Neves (1991)"

For analyzing the forces on anchor a few assumptions must be made. First, the soil type is limited to granular and in this way the effects of cohesion and adhesion could be neglected. As a next assumption the anchor will be penetrated the soil in low speed manner. Therefore inertia and water tension can also be excluded. Finally, the force acting on the fluke point will not be considered because it will be fully cancelled by the force perpendicular on the shank.

As illustrated in Figure 5 the anchor is considered to be subjected to four lumped forces: the chain force, forces on the front and back (or tip) of the fluke, and a force on the shank. The soil wedge is acted on by the force at the front of the anchor fluke, the shank force, and frictional resistance at the front and sides of the wedge (Neubecker and Randolph 1996a).

Some of the problematic mentioned forces are described in the following section:

1. Chain force (T_a): In order to determine the value of chain tension at the anchor pad eye, Neubecker and Randolph (1995) made some simplifying assumptions and obtained the closed form solution to the anchor line equations shown here.

$$T_a = \frac{T_0}{e^{\mu\theta_a}} \qquad [1]$$

Where T_0 is mud line load, μ is the friction coefficient (approximately 0.4), and θ_a is the inclination angle (approximately 20 in degree).

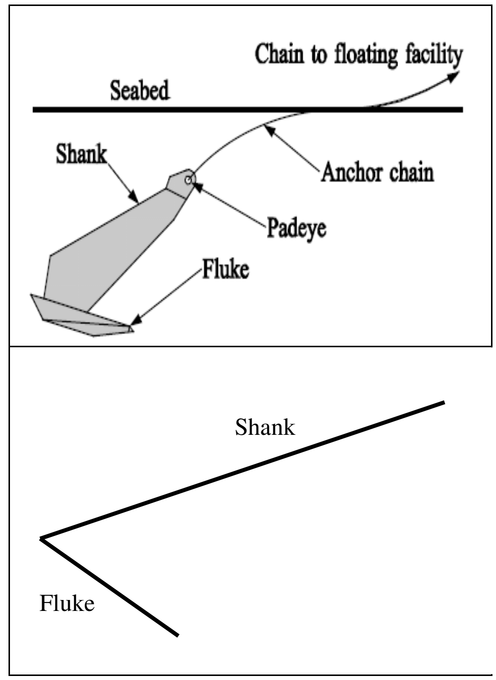

Figure 4. Drag anchor and chain system with the simplified form

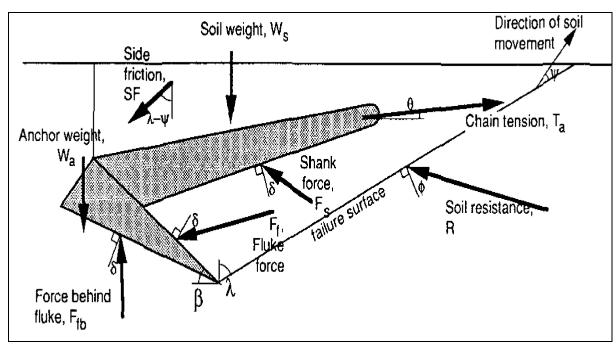

Figure 5. Force system incorporating force behind fluke proposed by Neubecker and Randolph (1995)

For assessing the mud line load (T_0), some correlations have been developed by various researchers. One of the most common correlations that assume the ship in static condition is as follows:

$$F_{XW} = 0.5 \times \rho_a \times V_W^2 \times A_X \times C_{XW} \times f_{XW}(\theta_W) \qquad [2]$$

Where F_{XW} is longitudinal long force and is equalled to T_0 (N), ρ_a is mass density of air, V_W is wind speed (m/s), A_X is transverse wind area of the ship (m^2), C_{XW} is longitudinal wind force drag coefficient, $f_{XW}(\theta_W)$ shape function for longitudinal force, and θ_W is wind angle (degree).

According to the some reasonable assumption based on typical ships and wind speed in the Persian Gulf, F_{XW} is calculated and is equal to 560 kN.

2. Shank bearing capacity (F_s): It has been observed that the motion of the anchor relative to the failing wedge of soil is such that the shank is continuously cutting through the soil (Neubecker and Randolph 1996). This fact is demonstrated by Carchedi (1984) that the force on the shank depends on its size and shape; so, the shank force should be calculated from the viewpoint of bearing capacity. The following relation is suggested for calculation of this force:

$$F_s = A_s \times \gamma \times d_s \times N_{qs} \qquad [3]$$

Where, A_s is the area of the shank, d_s is the average depth of the shank, and N_{qs} is the bearing capacity factor for the shank. It is demonstrated by Neubecker and Randolph (1996b) that N_{qs} is a critical unknown in the anchor equilibrium and its effect on shank resistance should be minimized in order to obtain good penetration and holding capacity. Therefore, the effect of shank force on the anchor has been ignored in this study.

3. Force behind fluke (F_{fb}): Existence of a force that acts on the back of the fluke is proved by theoretical considerations and physical experience. Although, the direction of this force is recognizable, its amount would be variable; because, this is a function of penetration's depth.

4. Side friction force (SF): According to the three-dimensional soil failure wedge (Figure 6a) that is an idealized failure mode for the soil, the side friction force is equalled to:

$$SF = \frac{\gamma L (H+h)^2 (\sin\phi - \sin\psi)}{4\cos\psi (1 - \sin\phi \sin\psi)} \qquad [4]$$

Where, Figure 6 illustrates the definition of some variables, L is fluke length, ϕ is the friction angle of the soil and ψ is the inclination of failure surface of straight line that is equal to dilation angle.

For determining the behind fluke and Side friction forces that is applied to the anchor, firstly, the geometry and weight of anchor should be selected. According to the chain force and catalogs of different anchors, the weight and dimension of the anchor have been selected as follows:

Shank length=2.5m
Fluke length=2.0m
Width of anchor=1.0m
Anchor weight=3.5ton
The angle between shank and fluke=45 (in degree)

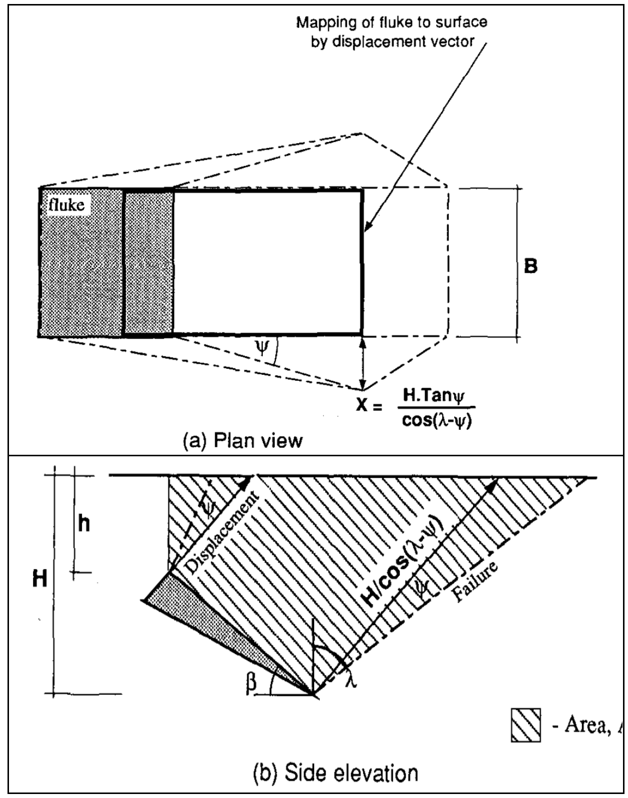

Figure 6. Three-dimensional failure wedge

4 DETERMINING THE HEIGHT AND WIDTH OF PROTECTION

In order to clarify the determining method for protection's dimensions, the possible conditions of anchor's collision with protection should be studied firstly. To design the geometry of protection two prominent principles should be considered: 1) anchor should not penetrate in protection layer and 2) the failure surface that will be induced by anchor penetration should not pass from the pipelines area. These principles have been used for determining the height and width of protection respectively. Based on the mentioned points, penetration's depth, angle of failure surface and maximum mobilized force in this surface, would be determined by iteration solving of forces polygon on anchor and soil in protection and seabed materials.

4.1 Possible Conditions of Anchor's Collision with Protection

Collision of anchor with protection could be happened in three different conditions that are reviewed herein:

1. The anchor could be collided with the protection's surface (Figure 7a). Based on the first principle, in this situation no penetration will be happened.

2. The anchor could be collided with seabed exactly on conjunction of protection and seabed (Figure 7b). In this situation, the protection layer should satisfy the minimum width and height that is required for preventing of anchor's penetration. The protection's height is calculated by solving the forces polygon that is formed in this condition that the failure wedge will be completely formed in the protection material. In order to design protection width accurately, it should be more or ultimately equal to width of failure wedge. It should be noted that this value for protection's width will be compared to width value that obtain in the third condition to conclude reasonable result.

3. The anchor could be collided with seabed some meters before protection (Figure 7c). In this situation, the chain force (T_a) could only mobilize in either the seabed material or both seabed and protection materials. The most critical and conservative condition is that the maximum penetration is achieved in seabed material and the movement of anchor is halted in the nearest condition to the pipelines. Because of the sufficient height of protection that is obtained in the last stage, the anchor could not drag under the protection area. This is a critical point that is named Q in Figure 7c. As mentioned above, the width of protection is the maximum of width failure wedge in second and third condition.

4.2 Forces polygon: the iteration solution

Below steps should be followed:

1. Assuming the depth of penetration and failure surface angle (λ).

2. Solving the soil polygon (Fig 8a) for different failure's angle from 1 to 90 degree and determining the amount of F_f and R forces.

3. Solving the anchor polygon (Fig. 8b) by using of F_f that is calculated in the last step to determine T_a and F_{fb} forces.

4. The failure wedge angle, λ, is then varied and the process repeated to obtain a minimum upper bound estimate of T_a.

5. If $T_{a(\min)}$ is less than the chain force (T_a), the depth of penetration should be increased and go to second step. Otherwise, the assumed depth will be considered as maximum depth of penetration.

6. Height and width of protection is determined by following equations:

$$H_p = H_f \quad [5]$$

$$W_p = \max\begin{cases} H_f \tan \lambda_f \\ H_n \tan \lambda_n \end{cases} \quad [6]$$

Where, H_p and W_p are the height and the width of protection layer respectively, H_f and λ_f are the embedment's depth and the angle of the failure surface of anchor in protection material, and H_n and λ_n are the depth of embedment and the angle of the failure surface of anchor in natural soil.

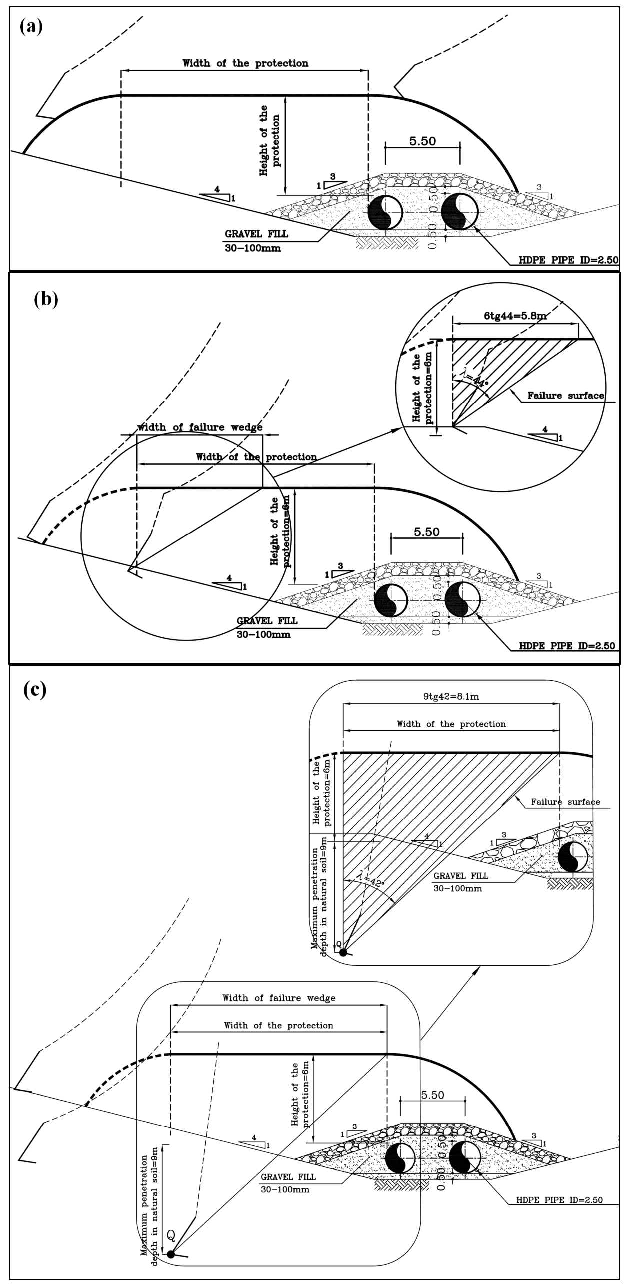

Figure 7. Anchor collided with seabed on the protection, exactly on conjunction of protection and seabed, and some meters before protection.

4.3 Results

Figure 9.a and 9.b illustrated the angle of failure surface and $T_{a(\min)}$ in filter and natural soils, respectively. Moreover, the value of the depth of embedment is added in caption of these figures. According to calculation results for filter material, the height that is needed is equal to 6 m. Moreover, the angle of failure surface that is formed through the protection is about 44 degree; Therefore the width of failure wedge will be equal to $6\tan(44)=5.8\text{m}$.

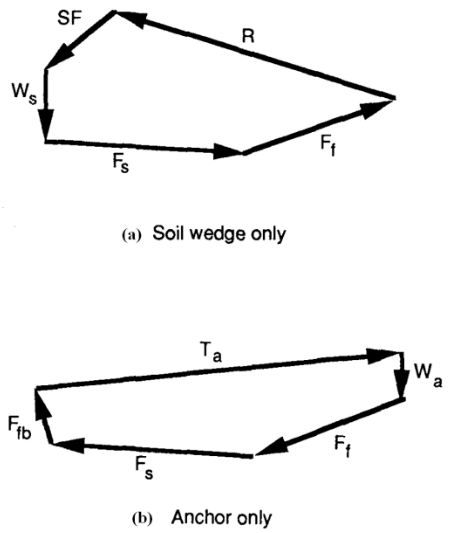

Figure 8. Force polygon for different free bodies

In this situation, the depth of anchor penetration and the angle of failure surface, in accordance with calculation results that are shown in Figure 9b, are equal to 9 m and 42 degree respectively. Therefore the failure surface that is intersected Q has a width equal to $9\tan(42)=8.0\text{m}$. Therefore 8 m is selected as protection width in conservative way.

According to the result of calculation, Table 3 is denoted the geometry of protection layer.

Table 3. Characteristic of pipeline protection

Geometry	Height	6 m
	Width	8 m
	Length	300 m
	Approximate volume	20000 m^3
Material	Saturated density	1900 kg/m^3
	Initial friction angle	41
	Cohesion	0

5 CONCLUSION

This report has based on a limit equilibrium approach to estimating the geometry of the protection layer with the viewpoint of holding capacity of drag anchors in cohesionless soils. The approach is originally based on Le Lievre and Tabatabaee (1979, 1981)'s recommendation, but is extended by Neubecker and Randolph (1996a) to incorporate a more realistic three-dimensional failure pattern in the soil, as well as the force acting on the back of the fluke. This approach has been implemented for designing pipe line protection in LNG port of Iran; although, physical model have been occasionally used for this purpose. By using of this approach, in addition to achieving logical results, the cost and time of project has been saved.

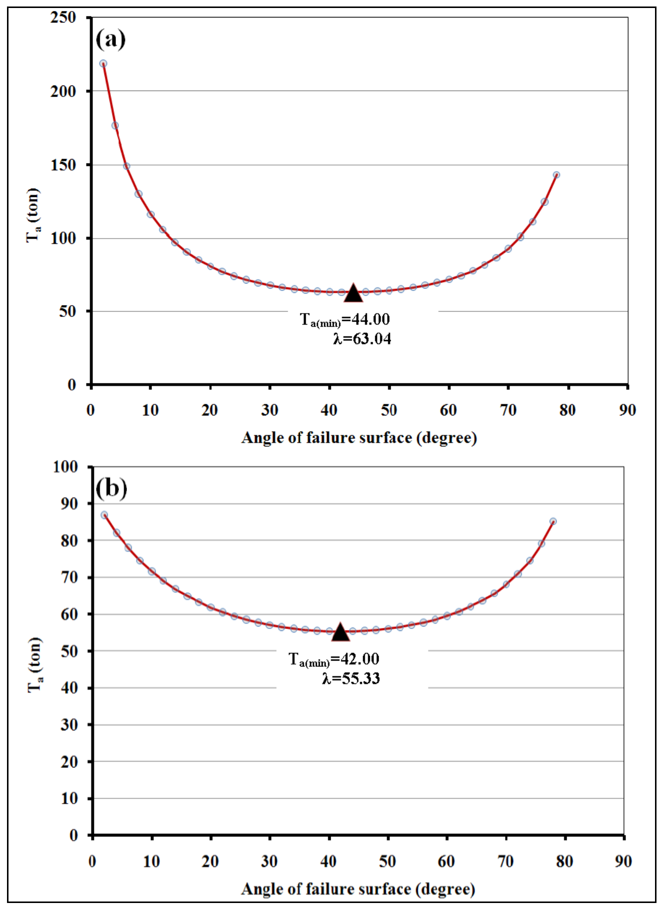

Figure 9. Variation of the failure angle to achieve minimum chain force (T_a) at the maximum penetration depth (6 m) in filter material (a) and (9 m) in natural soil (b)

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