MC252 Demobilization Project

for

MC252-1

Abandonment Plan for Coffer Dam - Supplemental Information to "Temporary Installed Subsea Equipment Recovery Plan (Procedure 4746)"

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1 Introduction

During the MC 252 incident response, BP installed several temporary pieces of equipment in the water column and on the seabed for the purpose of containing flow from the well, monitoring the well, monitoring subsea work activities, and ultimately killing the well. One containment strategy that was employed was placement of a Pollution Containment Chamber, or Coffer Dam, over the well in order to direct flow into a top hat device and containment riser to the surface. Build-up of hydrates within the structure prevented containment fluid from being properly directed through the top. The Coffer Dam was subsequently wet-parked on the seafloor at approximately 1,600-ft west of well MC252-1.

In this document, BP is proposing, for BOEM approval, that the coffer dam be abandoned in-place where it presents no risk to safety, the environment, or existing infrastructure.

An image of the Coffer Dam before installation is show below in Figure 1.

Figure 1: MC 252 Macondo Coffer Dam Load Out Before Installation

2 Coffer Dam Overview

The coffer dam containment device is parked 1,600-ft to the west of the BOP area. It is approximately 23.5-ft long X 13.5-ft wide (excluding mudmat-wings) and 41-ft high. The coffer dam is located on the seabed in 4,980-ft water depth. The structure is leaning approximately 20-degrees and embedded 6 - 12-ft into the seabed and secured to a hold down anchor. In its current configuration the coffer dam protrudes 32-ft above the seabed. Due to its location and water depth, it presents no hazard to navigation and commercial fisheries on the OCS. Its location is accurately known and can be annotated on shallow hazard maps for the area.

The internal volume of the coffer dam is 10,787-cu. ft. During the attempted installation over the leaking well, hydrates formed within the coffer dam upper "hat" section above the mudmats, on the outside of the structure, and on the mudmats. Hydrates that formed on the outside of the coffer dam were removed during the attempted installation operations; however, the estimated volume of hydrates remaining within the coffer dam is 6,218-cu. ft.

Due to the size of the device and the presence of hydrates in the assembly, it presents a significant handling risk. Buoyancy forces as a result of hydrate formation create uncertainty in center of gravity calculations used for lift planning. This and potential suction forces resulting from the embedment would create unpredictable and correspondingly dangerous conditions for support vessel personnel if a lift were attempted.

Any attempt to recover the coffer dam would first require dissociation of the hydrates prior to lifting it near the surface or out of the water. The volume of methanol required to dissolve the hydrates within the coffer dam is too large for practical application. In addition, preliminary estimates indicate that dissociation would occur at a water depth of approximately 300-ft. This poses a significant safety risk to a recovery operation since all of the hydrates in the coffer dam would attempt to dissociate at once in the water column in close proximity to the support vessel. Approximately 1.1 million-cu. ft. of methane would be release almost instantaneously to the surface in the immediate vicinity of the work vessel. Such a large volume of methane presents a fire / explosion risk for the support vessel. There is also the risk of the large volume of methane undermining the support vessel causing a stability risk to the vessel. In addition, hydrodynamic jetting effects of the dissociated gas escaping the interior of the coffer dam would present unpredictable dynamic loading on any down line and tag line used to control the lift. These dynamic loads could result in loss of control of the load, rigging failure or the load becoming unhooked during the lift.

The coffer dam has been inspected to verify its location and on-bottom stability and it is recommended that it be abandoned in-place where it presents no risk to safety, the environment, or existing infrastructure.

3 Long-Term Stability Analysis Results

The long-term stability analysis was performed by GEMS (Geoscience Earth & Marine Services, Inc.) and is provided as Attachment 1. The study assesses the foundation stability of the Coffer Dam as it currently rests on the seabed with regard to sliding, torsion, overturning, and further penetration. Coffer Dam geometry in the seabed, 100-year hydrodynamic loading, and area-specific soil conditions were used as factors in the stability analysis. Various possible failure modes were assessed, taking into account several uncertainties in parameters. The analysis concludes that the Coffer Dam has an acceptable Factor of Safety (FOS) against overturning and other forms of movement.

Attachment 1: Analysis of the Foundation Stability Macondo Pollution Containment Chamber (Cofferdam #1) Mississippi Canyon Area, Block 252, Gulf of Mexico, GEMS Report to BP, 15 November 2010.

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November 17, 2010

Attention: Mr. Martin Pabon
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ANALYSIS OF THE FOUNDATION STABILITY
MACONDO POLLUTION CONTAINMENT CHAMBER (Cofferdam #1)
MISSISSIPPI CANYON AREA, BLOCK 252, GULF of MEXICO

INTRODUCTION

BP requested this study to assess the foundation stability of the cofferdam that was used during the oil spill and was set down on the seabed approximately 1600 feet away from the well site. BP plans to leave the cofferdam on the seabed as it currently rests. The cofferdam penetrated into the seabed at an angle and BP has asked GEMS to assess its stability with regard to sliding, torsion, overturning and further penetration. The soils at the site consist of very soft to soft clay. Plastic limit analysis using simplified models was used to determine the suction caisson capacity in the possible failure modes.

PROBLEM DESCRIPTION

In this section, we summarize the conditions selected for analyzing the cofferdam including the caisson geometry and present condition, the design loads, and the soil conditions.

Cofferdam Geometry

The cofferdam is rectangular in cross section. It is approximately 23.4 feet long by 13.4 feet wide by 31 feet high (edges extend approximately 2.25 feet above the top) as shown in Figure 1. A pyramidal dome sits on top of the 31 foot high section with the top of the pyramid approximately 39.5 feet above the base. The cofferdam is open at the bottom and has a door cut out on one narrow side and on one wide side. The base of the cofferdam bearing on the soil is a tubular member approximately 8 inches wide. The interior of the cofferdam is lined with vertical "T beams" to stiffen the sides.

A picture of the current position of the cofferdam is shown in Figure 2. The cofferdam exhibits major tilt of 23 degrees in the broadside direction with a small tilt of 2.4 degrees in the end on direction. For the purposes of this analysis we will consider only the major tilt (broadside direction) as the small out of plane tilt is deemed to have a minimal effect on the stability. A simplified schematic of the configuration analyzed is shown in Figure 3. The submerged length of the broadside edges is taken as the average of the corner values as shown (12.5 feet on the deep side and 6.5 feet on the shallow side) and the submerged length of the centerline on the narrow face is then taken as 9.5 feet.

Load Conditions

The loads on the cofferdam are shown schematically in Figure 3. The cofferdam dry weight was reported to be 172 kips, resulting in a weight in water of 149.6 kips. Since the caisson is partially submerged in soil, its effective weight is further reduced by approximately 6.6 kips to give a total of 143 kips. The center of gravity is approximately 18.2 feet above the base. The lateral load from the weight because of the tilt is then 55.8 kips giving an overturning moment about the base of 1016 ft.-kips.

The hydrodynamic loads due to the 100 year current (0.8 knots) were estimated by Mr. Jim Adamson (Ref 1) for current directions in increments of 45 degrees around the cofferdam. These loads are very small relative to the overturning component due to its submerged weight, but they constitute the only net horizontal load on the cofferdam (weight and hydrate loads are strictly vertical and produce overturning/righting components only due to the cofferdam tilt). For simplicity we assumed the largest lateral load and resulting moment from the current occurs in the worst direction i.e. coincident with the overturning load from the weight. This is taken as a horizontal load of 1.25 kips at a height above mudline of 12.5 feet. This gives an overturning moment of approximately 27 ft-kips about the base of the structure.

The third load component is a result of the hydrates that have formed in the cofferdam. The volume of hydrates has been estimated to be 6218 cubic feet located in the upper part of the cofferdam (Ref 2). For our analyses we neglected the hydrates formed under the mudmats or wings attached to the structure since they appear to be relatively small compared to the volume within the cofferdam. The possible effect of these exterior hydrates is considered as part of the overall uncertainty in buoyancy. In this case the dome volume is approximately 950 cubic feet. The remainder of the hydrates would extend down approximately 17 feet below the base of the dome to give the estimated volume. There is, of course, uncertainty in this estimate. For example the maximum hydrate volume is deemed to be 7692 cubic feet which is the volume that would fill the cofferdam to the mudline or approximately 21.5 feet below the base of the dome. This conclusion is based on the fact that large resistance to penetration which would stop penetration would be encountered if the hydrate mass in the cofferdam contacted the mudline. Further, the hydrate specific gravity has been estimated to range between 0.7 and 0.9 (Ref 2). The center of buoyancy of the hydrates is estimated to be 24.2 feet above the structure base for the 17 foot case and 21.9 feet for the 21.5 ft case. This range of parameters obviously results in a rather large range of possible conditions. The strategy for selecting the appropriate value(s) will be discussed below in the section describing the analysis.

Soil Conditions

There are no site-specific borings at the site, so BP provided soil data from nearby sites (Horn Mountain and NaKika fields) which are believed to be representative of the site in question. A combined plot of these data is shown in Fig 4. The strength values in the upper ten feet of the profile exhibit significant scatter so a range of values will be considered. Specifically we considered in our analyses the following profiles

Profile 1:

$Su = 55$ psf $0 < z < 10$

$Su = 55 + 6.75 * (z-10)$ psf $z > 10$

Profile 2:

Su= 65 psf $0 < z < 10$

Su = 65 + 6.25 * (z-10) sf $z > 10$

Profile 3:

Su= 75 psf $0 < z < 10$

Su = 75 + 5.75 * (z-10) psf $z > 10$

The above strength profiles do not represent bounds on the values but rather include what we believe are lower (profile 1) and upper (profile 3) *plausible* values as well as best estimate values (profile 2).

A soil sensitivity value of 2.5 and unit weight of 33 pcf was used to assess the penetration resistance.

STABILITY ANALYSIS

As demonstrated above there are significant uncertainties in the analysis conditions with the primary uncertainties being the soil strength profile and the hydrate volume and specific gravity. To address these uncertainties we considered a range of values. The specific strategy employed was to conduct a series of penetration calculations for each of the soil profiles described above for both of the hydrate volumes (6218 and 7692 cubic feet), that is, we considered six cases. For each case we determined the hydrate specific gravity that was most consistent with the observed penetration to ensure that it was within the expected range (0.7 to 0.9).

For example what hydrate specific gravity for the assumed volume gives a net vertical load (weight in water of the cofferdam minus hydrate buoyancy) resulting in a driving force (parallel to the cofferdam centerline) equal to the soil resistance Using the specific parameters found in each analysis we performed overturning stability analyses to assess the factor of safety against overturning for that set of assumed conditions. Then we determined whether these factors of safety were in an acceptable range. This evaluation is considered the primary issue addressed in this study. In addition we demonstrated that the cofferdam was safe against further penetration as well as sliding and torsion failures.

Penetration Analysis

For the purposes here the cofferdam was assumed to have penetrated parallel to its current centerline. The resistance to this penetration was taken as the remolded soil strength integrated over the exterior surface area plus the undisturbed strength acting in end bearing on the 8 inch tubular around the base of the structure. Because of the large inset above the tubular it is our opinion that any internal skin friction will be small and was hence ignored. The area and base length missing because of the doors in the two sides reduces the resistance somewhat and is taken into account in the resistance calculation. For each soil profile used, the total resistance was estimated for the present position of the cofferdam. For the assumed hydrate volume, the specific gravity was determined that is consistent with the upward buoyant force required to make the net driving force equal the soil resistance, i.e.

$$R_s = (W_c - B_h) * \cos(\theta)$$

where R_s= soil resistance parallel to center line, W_c = in water weight of cofferdam, B_h = upward buoyant force of hydrate, and θ = tilt angle of cofferdam. A summary of the results of the six cases is provided in Table 1. It is important to note that the inferred hydrate specific gravity values fall well within the range estimated at the outset. These values were then used in an analysis of overturning as described below.

Overturning Analysis

For each of the six cases described above an analysis of overturning resistance was carried out. For this purpose we employed plastic limit analysis to estimate the failure load. Figure 5 shows the soil failure mechanism assumed in this analysis which is a simple slip circle centered on the cofferdam centerline a distance l_o above the bottom of the cofferdam. The circle is constrained to pass through the two bottom edges of the cofferdam. For each case we considered, the hydrate buoyancy and the current force were taken as known quantities. The parameter l_o was then varied to find the value that yielded the minimum cofferdam weight that would cause overturning failure.

Details of the governing equations are given in the Appendix. This mechanism has been shown by Bransby and Randolph (Ref 3) to give a good estimate of overturning stability for shallow foundations. To account for the finite length of the foundation, i.e. end effects, the solution is multiplied by a standard shape factor, (1 +0.2 (W'/L)), where W' is the width of the cofferdam intersection at the mudline and L is the length. The minimum possible l_o value is the value where the slip circle just intersects the cofferdam on the shallow side at the mudline. It turns out that the minimization procedure self selects this minimum value in all cases analyzed, i.e. this specific mechanism gives the minimum weight that will result in an overturning failure. The factor of safety (FOS) is then taken as the ratio of the calculated weight at failure to the actual submerged weight. The results of the analyses are given in Table 2 for each of the six cases. Note that the minimum factor of safety is 1.77 with a maximum of 2.00 depending on the assumed conditions. Thus for a wide range of assumptions the safety factor is relatively insensitive as long as the overturning calculations and the penetration calculations are self consistent. The solution is thus robust and the FOS's are acceptable in our opinion.

Analysis of Other Failure Modes

As mentioned above this section addresses three other possible failure modes: additional penetration, sliding, and torsion. The basic method of analysis is again plastic limit analysis.

Additional Penetration: While additional penetration does not seem to pose a significant risk, we analyzed it to assess the likelihood of its occurrence. We first considered two possibilities depending on the hydrate volume. If the hydrates filled the cofferdam to the mudline, then additional penetration was highly unlikely due to the abrupt increase in end bearing capacity. For the lower volume of hydrates the primary stabilizing force is soil set up.

In our opinion full soil set-up has most likely occurred on the exterior sides of the caisson. Thus the skin friction has increased from the remolded value (40 percent of insitu strength) to the fully set up value which, using API methods, is estimated to give an average alpha value of 0.76 for the lower strength profile. The end bearing on the bottom edge is taken to be the same as during original penetration; however, it would quickly increase with any additional penetration owing to the increasing strength with depth. For example, for Case 1, i.e. the lower strength profile and lower hydrate volume, the total resistance after set up is 52.4 kips compared to the initial

estimated resistance of 39.2 kips giving a FOS equal to 1.34. We consider this value to be satisfactory considering the initial penetration is, in effect, a proof test and the fact that the consequences of failure in this mode are low.

Sliding: It is noted that the only force that tends to cause sliding is the hydrodynamic load with a maximum value of only 1.25 kips. The weight and hydrate buoyancy are strictly vertical loads. They have components perpendicular to the cofferdam centerline due to tilt but have no horizontal component. Thus a very simple calculation shows that the resistance is many times the current load. For example for a shear strength of 55 psf and an average depth of 9.5 feet and length of 23.4 feet the simple Rankine passive pressure is approximately 24.4 kips, again many times the actual load. Thus the likelihood of sliding is negligible.

Torsion: The center of mass for the cofferdam is offset along the long axis a length of 1.26 feet as shown in Figure 1(Ref 2). The load normal to the cofferdam centerline (55.8 kips) due to the weight and the tilt yields a torque of about 70.3 ft-kips. The maximum torque due to current loading is 3.5 ft-kips (Ref. 1) and yields a total worst case torque of 73.8 ft-kips. This assumes that the hydrate center of buoyancy acts at the centerline, i.e. does not contribute to or diminish torsion. For a soil strength of 55 psf and a penetration depth of 9.5 feet the torsion capacity, including resistance on the cofferdam base and active and passive values on the sides yields a torsional resistance of approximately 1400 ft-kips using the method described by Murff, et al (Ref 4). Therefore, we consider the potential for a torsional failure to be negligible.

Additional Comments: In addition to the intrinsic stability of the cofferdam in its existing condition, a gravity anchor has been attached to the cofferdam by a wire rope. The anchor consists of a mat that is 6.5 feet wide by 16 feet long filled with sand bags. The total basket weight with sand bags is reported to be 20.4 kips (Ref. 2). It is not clear whether the mat has penetrated significantly to mobilize passive resistance in sliding on the sides; however, a minimum lateral resistance is estimated to result from the soil-mat bond caused by consolidation. We estimate that resistance to be the total weight times a factor of 0.2 or about 4.1 kips. An optimistic estimate is simply the shear strength of the in situ soil, say 55 psf, as a lower value times the base area. The base resistance yields a total lateral resistance of 5.7 kips. Thus the lateral resistance on the gravity anchor is likely in this range and provides additional capacity to prevent overturning of the cofferdam. This additional capacity provides further margin of safety to the cofferdam stability although the cofferdam is stable without it in our opinion.

CONCLUSIONS

The purpose of our analytical study was to check the stability of the cofferdam near the Macondo well site as it currently rests on the seabed. We investigated the various possible modes of failure to assess its stability with regard to sliding, torsion, overturning, and further penetration. In our opinion the overturning mode is potentially the most critical. We identified the effects of several uncertainties in parameters essential to the analysis and investigated their influence upon our analyses. We found that if a consistent set of parameters is used within a likely range of values, then the cofferdam has an acceptable FOS against overturning.

Our report is released under the authority of James D. Murff, Texas P.E. No. 39010 and Alan G. Young, Texas P.E. No 37478. A signed and sealed original of this report will kept on file at the GEMS offices. We appreciate the opportunity to assist BP with this project. Please call us if we can be of further service or if you have questions about this report.

Sincerely,
GEOSCIENCE EARTH & MARINE SERVICES, Inc.®, F-4614

James D. Murff, P.E., Ph.D
Senior Consultant

Alan G. Young, P.E.
Vice President

REFERENCES

1) Adamson, J. E., "Current Loads: Wet Parked Cofferdam", Okeanos Report to BP, Oct. 27, 2010.

2) Smelser, A. "Cofferdam Input Checklist", Attachment to e-mail dated Oct. 25, 2010.

3) Bransby, M. F. and Randolph, M. F. (1998) "Combined loading of skirted foundations", Geotechnique, 48, No. 5, 637-655.

4) Murff, J. D., Aubeny, C.P., and Yang, M , "The effect of torsion on the sliding resistance of rectangular foundations", Int. Symposium Frontiers in Offshore Geotechnics, Perth, Nov. 2010.

Table 1. Results of Penetration Analysis

Case	Strength Profile	Hydrate Volume, ft^3	Hydrate Buoyancy kips	Center of Buoyancy, ft	Net Soil Resistance, Kips	Consistent Specific Gravity
1	Profile 1	6218	102	24.2	37.5	.743
2		7692		21.9		.796
3	Profile 2	6218	95	24.2	43.9	.766
4		7692		21.9		.810
5	Profile 3	6218	88	24.2	50.3	.784
6		7692		21.9		.826

Note: The specific gravity is determined that gives a net downward force equal to the net resistance/cos(θ).

Table 2. Results of Overturning Analysis

Case	Strength Profile	Hydrate Volume, ft^3	Factor of Safety Against Overturning
1	Profile 1	6218	1.93
2		7692	1.77
3	Profile 2	6218	1.96
4		7692	1.82
5	Profile 3	6218	2.00
6		7692	1.87

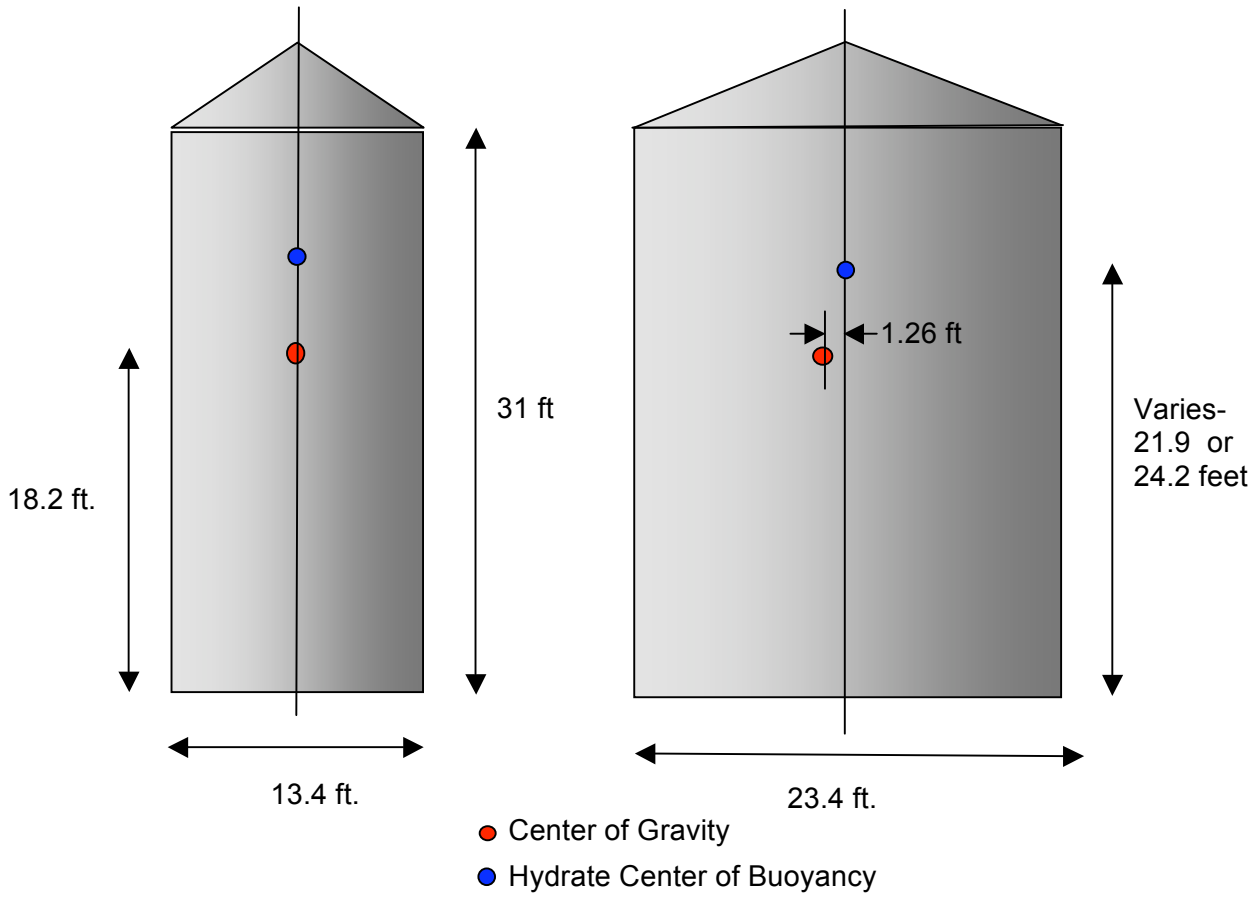

Figure 1. Schematic of Cofferdam – Basic Dimensions

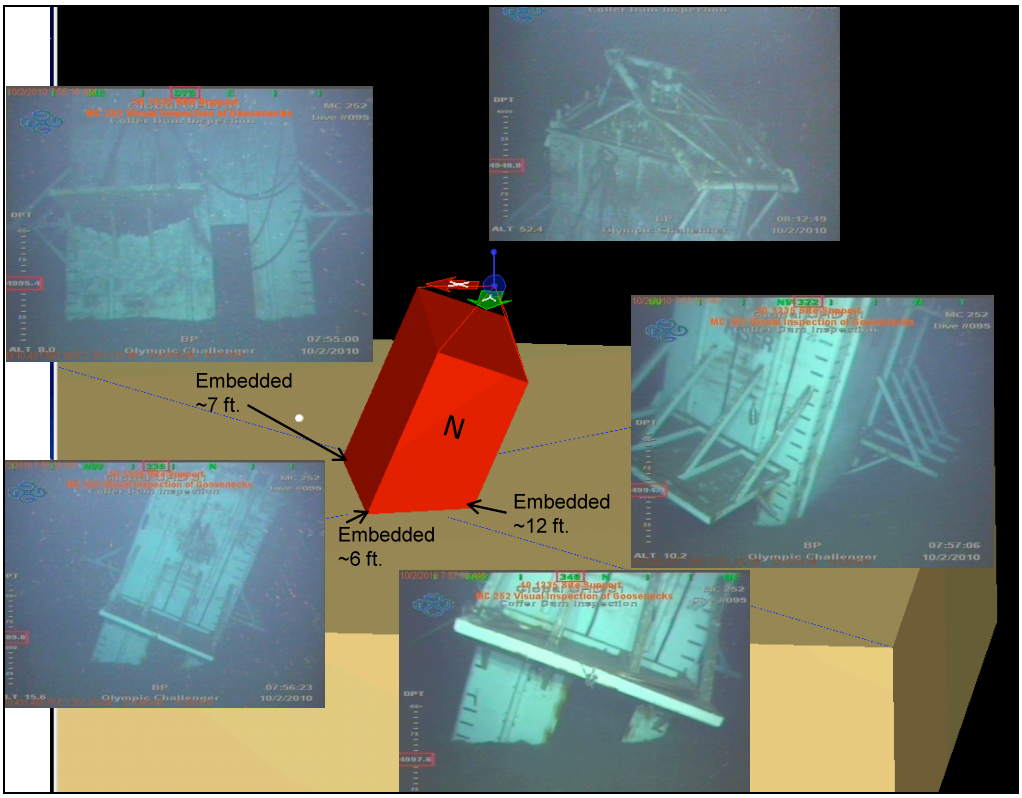

Figure 2. Photo of Cofferdam in Current Position

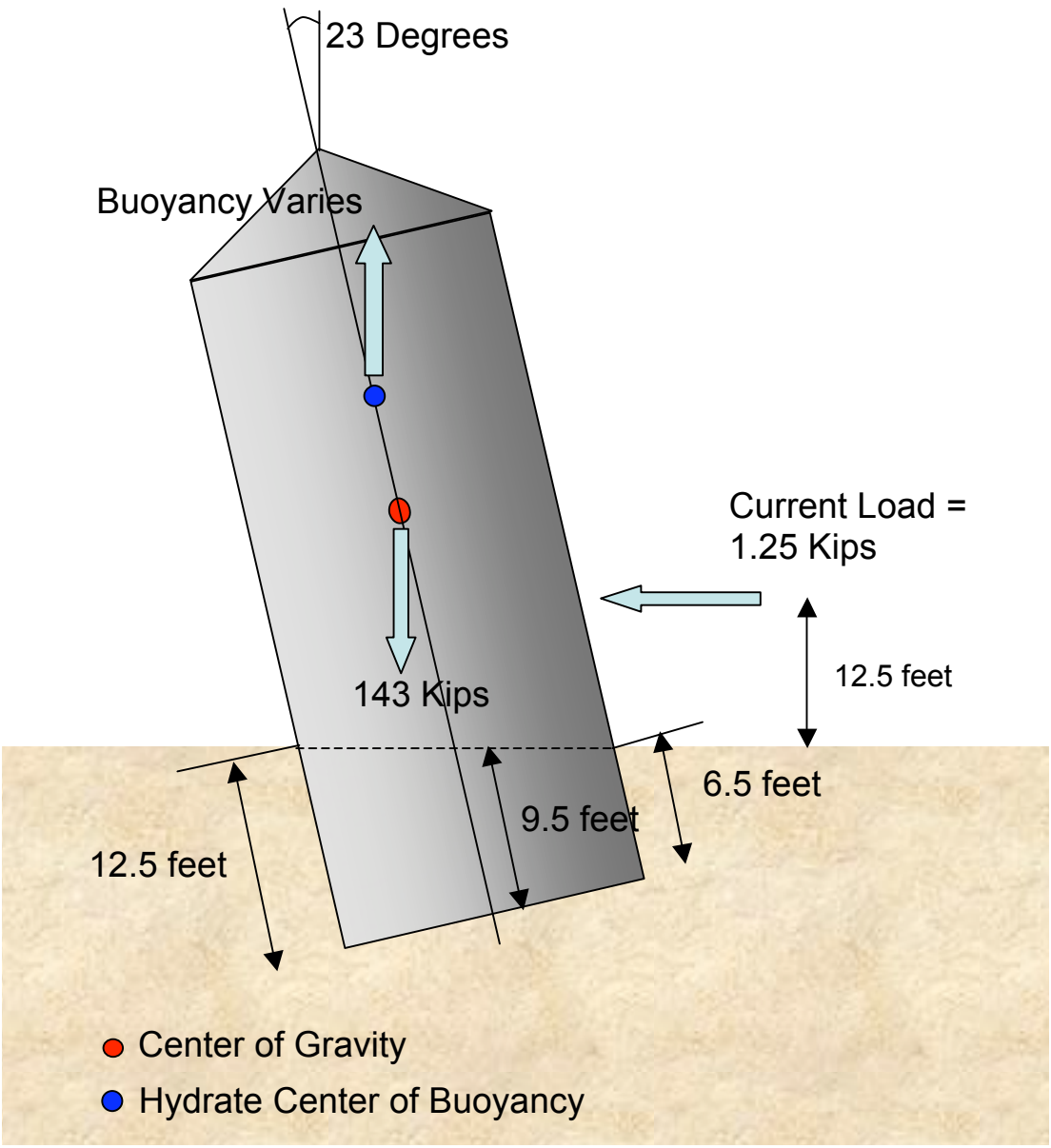

Figure 3. Schematic of Cofferdam - Current Position

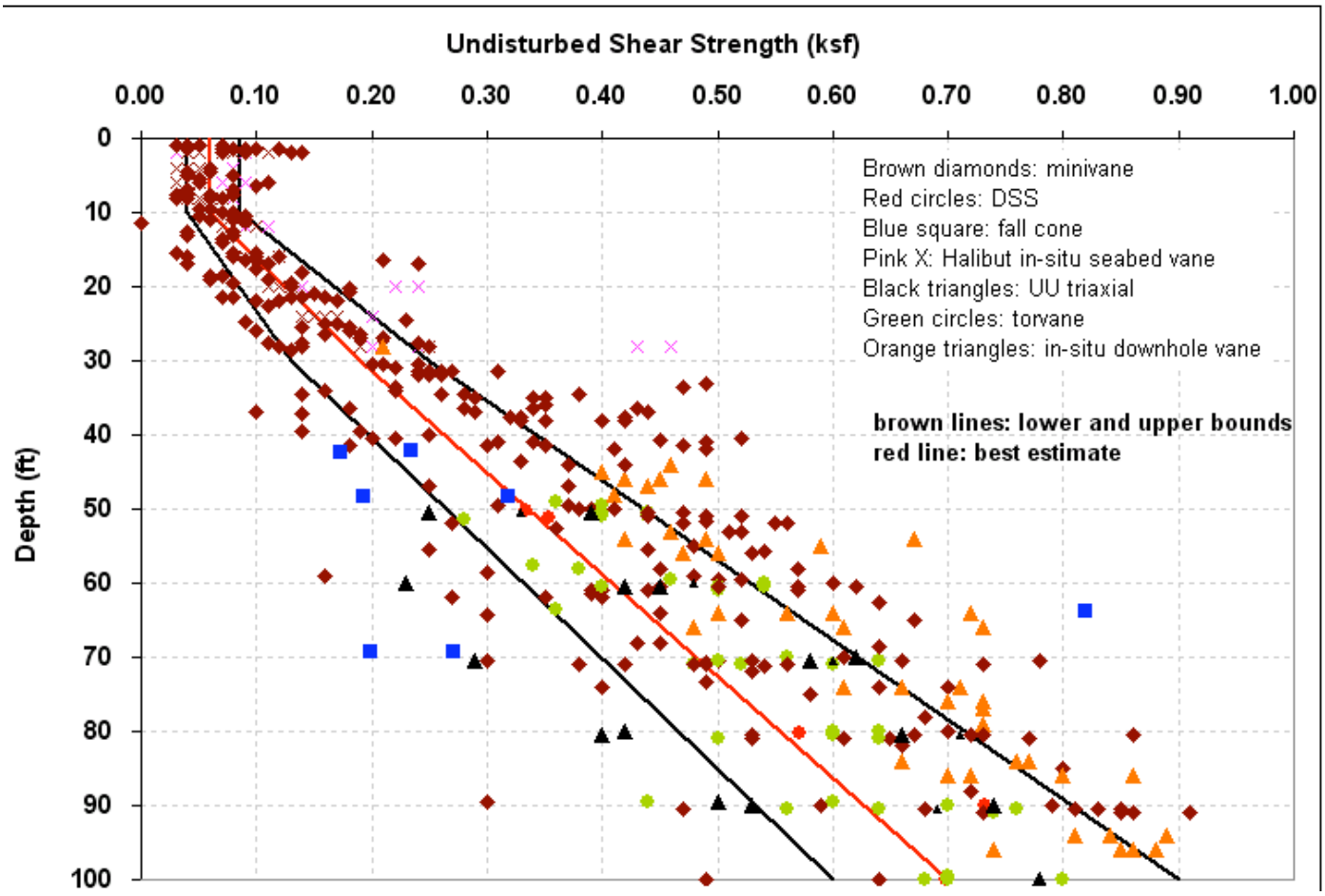

Figure 4. Shear Strength Data

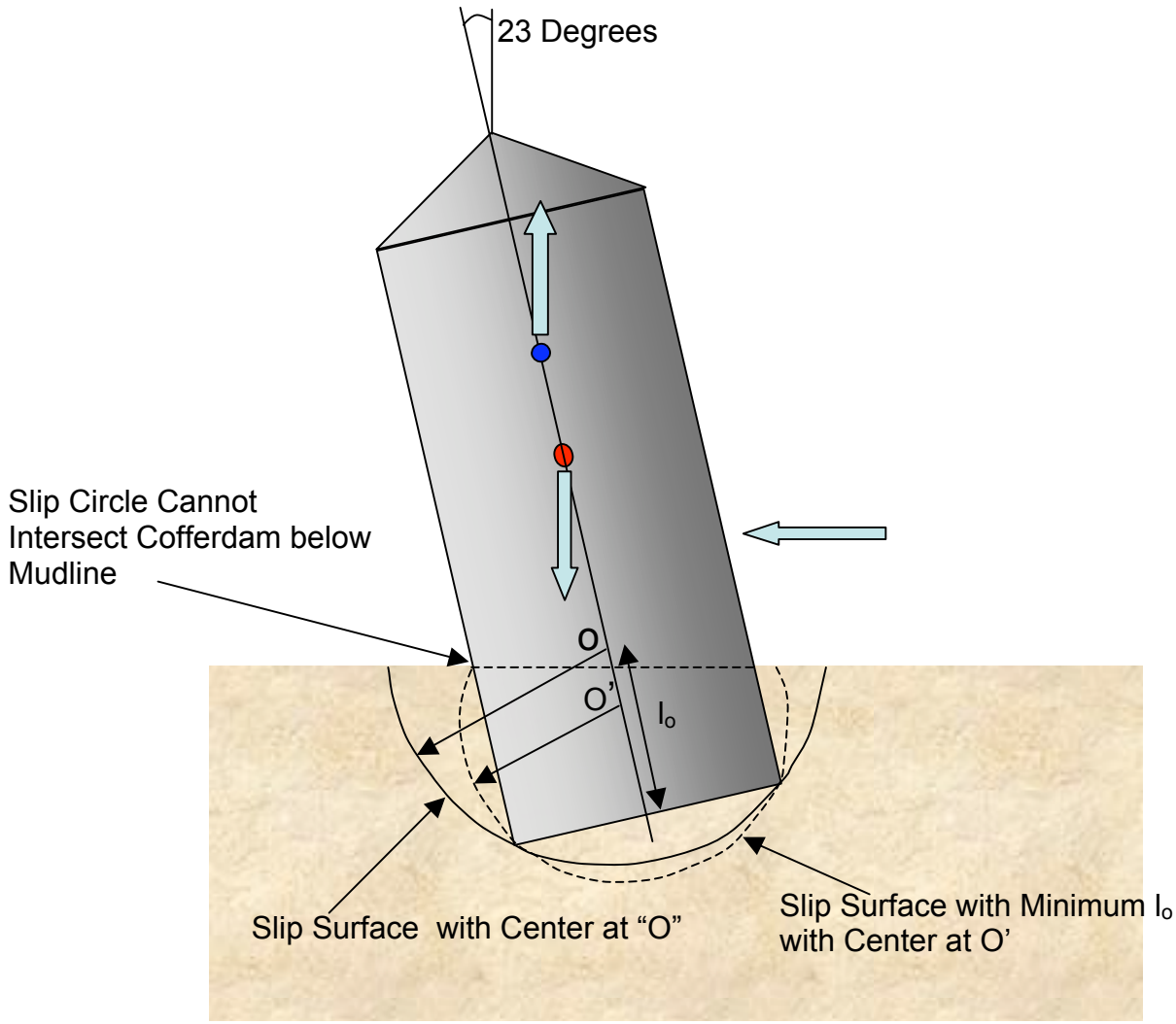

Figure 5. Slip Circle Idealization for Overturning Analysis

Appendix A. Derivation of Governing Equations for Slip Circle Analysis

In this appendix we will derive the governing equations for slip circle analysis of a shallow foundation as applied to the problem of the tilted cofferdam described in the text. Specifically we will apply the upper bound techniques which includes the following steps:

- Postulate a failure mechanism in terms of one or more unknown parameters
- Derive equations to calculate the rate of dissipation of energy in the yielding material due to a virtual velocity within the mechanism
- Find the virtual work rate done by the external forces, including the unknown force.
- Set the virtual work rate equal to the virtual rate energy dissipation and solve for the unknown force
- Vary the unknown parameter(s) to find the minimum force that will cause failure.

The mechanism selected for this analysis is a slip circle as shown in the Appendix Figure. The slip circle is centered on the centerline of the narrow side of the cofferdam a distance l_o (the unknown parameter) above the bottom of the cofferdam. The slip circle is constrained to pass through the two edges at the bottom of the cofferdam. This results in the slip circle radius, R_2 being

$$R_2 = \sqrt{l_o^2 + R^2}$$

The cofferdam and circular soil segment are assumed to rotate about the circle center, O, at an angular velocity of $\dot{\beta}$. The differential energy dissipation rate in the soil is then

$$d\dot{D} = soil\ strength\ x\ velocity\ x\ differential\ length$$
$$= s_u\ (z) * R_2\dot{\beta} * R_2 d\varphi$$

where s_u is a function of depth below mudline z and φ is the angle of a point on the slip circle measured from the horizontal. The total dissipation is then

$$\dot{D} = \dot{\beta}\int_{\omega}^{\pi-\omega} s_u(z) R_2^2 d\varphi$$

where

$$z = z(\varphi) = R_2 \sin\varphi - (l_o - p)\cos(\theta)$$

and

$$\omega = \sin^{-1}\left[\frac{(l_0 - p)\cos(\theta)}{R_2}\right]$$

Note that s_u is in general a function of z and hence the integration is numerical in the implementation.

The external work rate of the various loads (weight, buoyancy, and current load) is

$$\dot{W}_e = W_s * Sin(\theta) * (H_w - l_o)\dot{\beta} - B_h * \sin(\theta) * (H_B - l_o)\dot{\beta} + F_c * \cos(\theta) * (p + \frac{H_c}{\cos(\theta)} - l_o)\dot{\beta}$$

where parameters are as defined in the Appendix Figure.

Setting the external work rate equal to the internal rate of dissipation of energy, canceling the virtual rates (β's), and solving for the submerged cofferdam weight (W_s) that results in failure gives

$$W_s = \frac{D + W_B * \sin(\theta) * (H_h - l_o) - F_c * \cos(\theta) * (p + \frac{H_c}{\cos(\theta)} - l_o)}{Sin(\theta) * (H_w - l_o)}$$

W_s is then minimized with respect to l_o to get the best solution, i.e. the one nearest the exact solution.