INDEPENDENT REIVEW REPORT

EXCAVATION PROJECT, BLOCK 20 MISSISSIPPI CANYON AREA, GULF OF MEXICO

to
Taylor Energy Company
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March 1, 2008

SUMMARY

Drs. Robert G. Bea (Risk Assessment and Management Services of Moraga, California) and Harry H. Roberts (Roberts GeoServices of Baton Rouge, Louisiana) were contracted by Taylor Energy Company of New Orleans, Louisiana to act as independent reviewers of a geotechnical engineering Final Report submitted to Taylor Energy by Fugro-McClelland Marine Geosciences, Inc. (FMG) which deals with the feasibility of a large sediment excavation project in Mississippi Canyon Block 20 (MC 20) Gulf of Mexico. The purpose of the potential excavation is to expose well site conductors so that Minerals Management Service requirements to "plug" abandoned wells can be met. As a product of Hurricane Ivan in September 2004 the Taylor Energy Company production platform in MC 20, was toppled, moved over 500 feet downslope, and the well jacket buried approximately 150 feet beneath the mudline. In order to plug and abandon the wells one option is to expose the jacket by removing the sediment that now buries it. The Fugro-McClelland report addresses the feasibility of this option.

This independent review of Fugro-McClelland's geology and engineering analyses final report (FMG Report No. 0201-6235-2, 2008) is presented in two parts. Part 1 defines the geologic conditions of the project area and the geologic/geotechnical boundary conditions for evaluating the feasibility of the excavation project. Part 2 evaluates the project from the standpoint of reliability based factors-of-safety. The following key summary points for Parts 1 and 2 are presented as the abstracted conclusions of our independent review.

Part I: The Fugro-McClelland Final Report (FMFR) does a very credible job of presenting the geologic framework for the project. Since the loss of a platform on the Mississippi River delta-front during Hurricane Camille in 1969, the instability of continental shelf deposits offshore of the modern delta has been a subject of research and many types of data collection. Data collected as a part of the MC 20 study confirm that young and unstable sediments of the delta-front occur throughout the project area, not just in the immediate vicinity of the toppled platform. Several critical points are made from the geological assessment of the report (Appendix A).

(1) The deepest potential slideplane below the base of the "unstable mudflow and underlying thin debris flow deposits" is a stratigraphically isolated and thin debris flow unit 178 feet below the mudline in Boring 2. This debris flow deposit is bracketed by sediments that are from a different depositional setting and accumulated slowly, hemipelagic and thin turbid flow deposits. Horizon 5 (FMFR), determined from high resolution seismic data, defines the surface on which mudflow and an underlying series of thin debris flow deposits downlap. The debris flow deposit at 178 feet is clearly below Horizon 5. However, it is difficult to correlate seismic Horizons 4 and 5 of the FMFR with the base of the mudflow deposits and base of the composite debris flow section from Boring 2. A correlation error of 10 + feet exists. This error probably arises from estimating sound velocity through the mudflow-debris flow section and picking the debris flow unit that downlaps on Horizon 5. Although the 178 foot horizon in Boring 2 is a potential slideplane, the base of the excavation

(Factor of Safety, FOS = 1.0) is projected below the debris flow unit (a subsurface depth of 180 feet) at the well bay site.

(2) Along-shelf and cross-shelf seismic profiles clearly establish the complex stratigraphic architecture and interpreted corresponding variability in sediment characteristics in all directions away from the well bay site. Since the engineering borings supporting the investigation of the MC 20 project area are basically clustered around the pre-Ivan production platform location, it is doubtful that the Fugro-McClelland geotechnical database captures the true variability of sediment properties within the limits of even the most conservative excavation feature (FOS = 1.0). Using a "layer cake" geologic model (FMFR) to project soil properties over large areas does not account for the nested instabilities that certainly must occur in the complex of gullies and compensationally stacked mudflow lobes common to the project area.

(3) In evaluation of updip deposits that would be removed as part of the excavation feature defined by a FOS = 1.0, regional seismic data clearly show that dredging would extend into highly unstable deposits, some of which are more recent depositional units than the mudflows around the platform site and probably more prone to failure. It is important to note that failures naturally occur around the entire perimeter of the modern Mississippi Delta in association with bottom slopes of $< 0.5^{o}$. The slope of the excavation feature north of the well bay using a FOS = 1.0 is projected to be 1.3^{o}, almost three times the slope of unstable parts of the natural delta-front that consistently fail.

(4) If the sediment excavation around the well bay is completed, it is estimated that divers involved in preparing and plugging the individual wells will need approximately 400 days to complete the task. No analysis of potential storm wave impacts on wall stability of the excavation feature over this time period was discussed in the report. However, storm waves were mentioned as impacts to be considered.

(5) For an excavation constructed to a depth of 180 feet at the well bay using a marginal FOS = 1.0, an estimated 60×10^{6} yd^{3} of sediment must be removed. Disposal of this sediment is an enormous undertaking with strong environmental implications. Using a reasonable (minimal) FOS = 1.5, the results indicate that the excavation is not feasible.

Summary Statement: *The MC 20 project area is part of a continental shelf-slope province that is unmatched worldwide for delta-front sediment instability. The FMFR demonstrates a depth of knowledge and long history of dealing with engineering problems related to slope failures offshore of the Mississippi River Delta and at the shelf edge. Modeling slope stability in this depositional setting characterized by a truly complex stratigraphic architecture is not a trivial task. Even though weak layers in association with lower bound shear strength and upper bound unit weight of soils from near the well bay were used as design criteria for the excavation, the slope stability analysis probably does not account for the nested instabilities in this complex geologic system. However, using local soil parameters and* *horizontal stratigraphy (FMFR model) to predict slope stability over a large area clearly over simplifies the problem and leads to a truly optimistic result. In the marginally stable case (FOS = 1.0), the excavation may be technically feasible, but maintaining wall integrity for a sufficient period of time to prepare and plug 28 wells is probably not possible. Gravity-driven slumping and failure forced by possible storm wave interactions with these highly unstable sediments is probable during the 400 + day period estimated for divers to complete their work at the well bay site. Finally, even though FM was not tasked with the problem of sediment disposal, the enormous volume of sediment from the excavation is a substantial problem with environmental consequences. From a geologic point of view, the excavation modeled with a FOS = 1.0 carries a high risk of wall failure under both normal gravity-driven and wave-driven forces. The excavation area and volume modeled with a reasonable FOS = 1.5 is not technically feasible.*

Part II: The FMFR addresses the reliability aspects of the proposed excavation project in MC 20 in the form of variable FOS and the "upper and lower bound" soil characteristics used in the analyses to determine the FOS for given excavations. The FMFR does not explicitly address the reliability or risk (likelihoods and consequences of failures) aspects of the proposed excavations in MC 20. The FMFR does address several sources of information that provide insights into acceptable FOS for conventional excavations. As noted in Part I, the FMFR indicates for FOS in the range of those indicated as required for conventional excavations (e.g. FOS = 1.3) the excavation is not feasible. Several critical points are made from evaluation of the reliability implications of the report (Appendix B).

(1) Given that there are no implicit 'biases'[1] included in the analytical models employed in the FMFR, a FOS = 1.0 is equivalent to a Probability of Failure (POF) of POF = 50%. The relationship between the FOS and POF is a function of the types of distributions of the excavation demands (e.g. gravity stresses) and slope capacities (e.g. effective soil strengths) and the uncertainties and correlations associated with these parameters. For a FOS = 1.3, a reasonable estimate of the POF is POF = 25%.

(2) The FMFR analyses of slope stability are idealized representations of a very complex geologic and geotechnical environment. As noted in Part I, the fundamental slope stability analytical model ('layer cake') is not able to characterize critical geologic features in this locale such as multiple flow lobes and gullies. The limit equilibrium analysis model used to evaluate the stability of the slopes is a similarly simplified characterization of the stability of a rheologic – viscous material on variable slopes subjected to variable environmental conditions. Characterizations of the soil 'shear strengths' and 'effective unit weights' also introduces additional uncertainties which have been addressed with the use of 'upper' and 'lower' 'bounds' for these parameters. In addition, concerns for significant creep related slope movements (which could lead to creep rupture and subsequent turbidity flows), wave and current induced forcing, and other similar factors add other layers of uncertainty. Thus, the results contained in the FMFR

[1] Bias is defined as the ratio of the true or actual value to the nominal or computed value

should be interpreted as very rough ‘indicators’ of what will actually be encountered in the proposed excavations.

(3) The FMFR includes a review and summary of available excavation safety guidelines. This review indicates FOS in the range of 1.3 to 1.5.

(4) A risk based assessment of the required FOS was performed as a part of this review (Appendix B). This assessment identified FOS in the range of 1.4 to 1.7 if advantage were taken of ‘proof testing’ the slopes before commencement of diver operations inside the excavation. Without such proof testing, FOS greater than 2.0 would be required to provide adequate diver safety.

(5) Given that FOS = 1.5 were required, the FMFR results indicate that the excavation would not be feasible.

APPENDIX A

Assessment of Geologic – Sedimentological Boundary Conditions For Excavation Project, Block 20 Mississippi Canyon Area, Gulf of Mexico

Report to
Taylor Energy Company
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By

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March 2008

Geologic Framework for the Proposed Project

The Fugro-McClelland Marine Geosciences, Inc. (FMMG) final report to Taylor Energy Company addresses the "Geology and Engineering Analyses" for removing approximately 180 feet of overlying sediment to expose the well site conductors at the former Mississippi Canyon, Block 20 production-platform site. The project site is located at the shelf edge approximately 10.5 miles southeast of the modern Mississippi River Delta South Pass distributary (Figure 1). The region between the subaerial delta and the shelf edge is arguably one of the world's most unstable continental shelf and shelf edge regions. Early studies of the offshore Mississippi River Delta revealed the presence of gullies that seemed to radiate downslope from the major distributaries of the "birdfoot" delta, (Shepard, 1955). Through the study of U.S. Coast and Geodetic Survey bathymetric charts and acquisition of data on his own, Shepard (1955) suggested that these gullies were similar to features cutting across submerged delta-fronts building into lakes of Switzerland (Bouma et al., 1991). He concluded that these features were associated with submarine mass movement. Later and more definitive research by Coleman et al. (1980) used side-scan sonar to create overlapping images of the seafloor and for the first time view all of the elements of the complex delta-front submarine landslide systems that started near the Mississippi River distributary mouths and extended, in some cases, to beyond the shelf-slope break.

The urgency to understand mass movement on the Mississippi River delta-front grew as the exploration for and production of oil and gas moved out of the delta's bays and onto the adjacent continental shelf. Toppling of Shell's South Pass 70B platform during Hurricane Camille in 1969 propelled the sediment instability problem to the forefront and as a result, many geological, geophysical, geotechnical, and geochemical studies of delta-front sediments and slope failures followed (Bea, 1971; Bea et al., 1975; Sterling and Strohbeck, 1975; Whelan et al., 1975; Whelan et al., 1976; Roberts et al., 1976; Prior and Coleman, 1978; and Bea and Audibert, 1980). Cooperative studies between industry, government, and universities were initiated after the South Pass Block 70 disaster. The results of these studies provided the fundamental understanding of delta-front sediment instability and mass movement still used to evaluate the Mississippi River Delta offshore today. McClelland Engineers, Inc., later to become Fugro-McClelland Marine Geosciences Inc., played an important role in developing our current understanding of delta-front sediment instability processes. Now, as Fugro-McClelland Marine Geosciences, Inc. the tradition of work in this subject area continues and expands. Their coverage of the regional setting in the Mississippi Canyon Block 20 Excavation Project Report demonstrates an excellent working knowledge of the general geologic framework conditions for the excavation project study.

Summary Comments: *The engineers at FMMG have a long tradition of addressing sediment instability problems that plague man's infrastructure in the Mississippi River delta-front environment. The earlier McClelland Engineers, Inc. (ME) pioneered instrumentation and data collection procedures necessary to understand the unusual marine soils of the delta-front and predict their behavior under various forcing conditions. Above all other geotechnical companies, FMMG has the history of dealing with sediment instability problems on the delta-front. Their continually evolving body of work on this subject allows them to clearly understand the geologic framework for the MC 20 excavation project.*

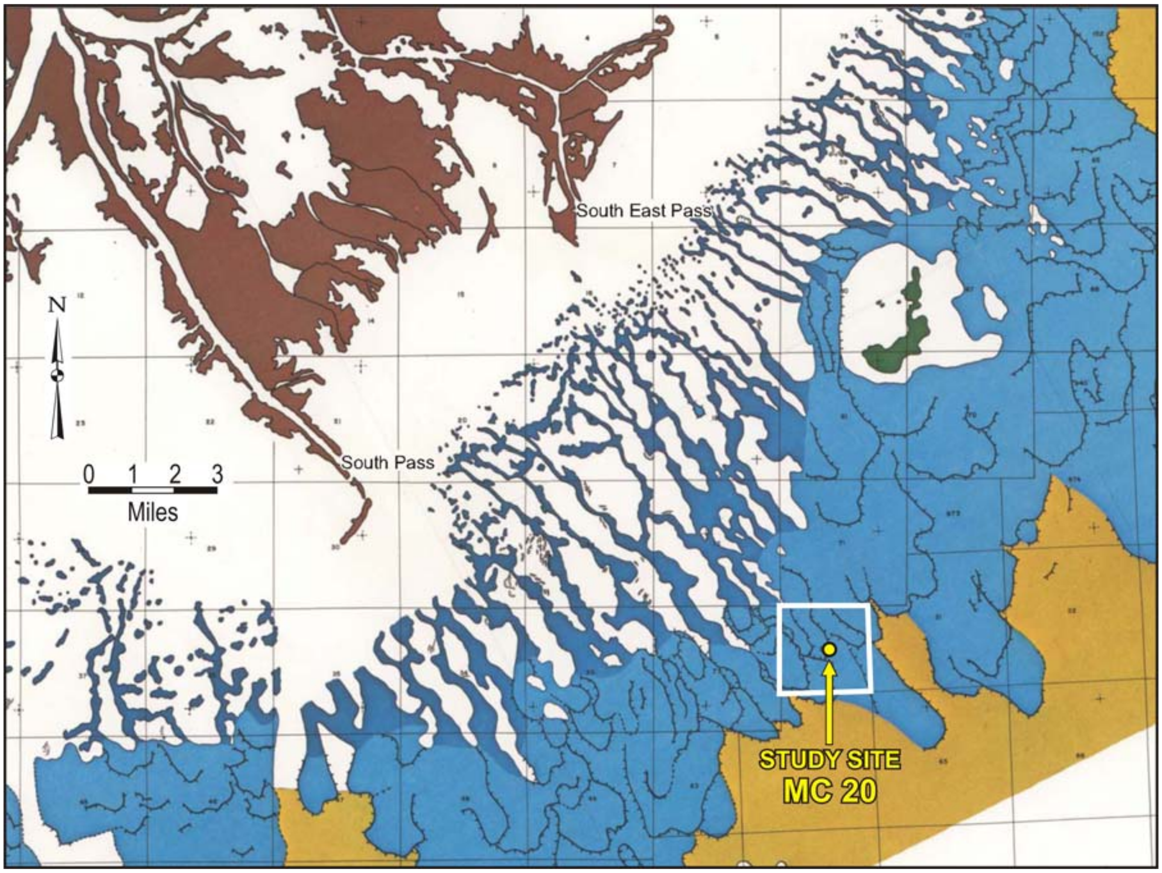

Figure 1. This map of the unstable Mississippi River Delta offshore, compiled from overlapping side-scan sonar data (Coleman et al., 1980), illustrates the array of gullies and associated downslope depositional sediment lobes common to the region. The gullies radiate from the perimeter of the subaerial delta, acting as sediment transport pathways to deeper water. Mississippi Canyon Block 20 is located at the shelf-slope transition in an area of overlapping distal mudflow lobes downslope and to the southeast of the delta's South Pass distributary.

Critical Geologic Characteristics of the MC 20 Site

Major slope failures and development of mudflow lobes on the mid-outer shelf and upper slope are linked to major storm wave interactions with a seafloor characterized by low shear strength sediments (Suhayda et al., 1976; Zhang and Suhayda, 1994). From continuing research which was initiated after the SP 70 platform loss during Hurricane Camille, it has become clear that cyclic loading by powerful tropical storm waves trigger seafloor failures in the delta-front's thick sediments which have very weak profiles. These failures pose a recurring threat to man's infrastructure on the delta-front and at the shelf edge (Coleman and Garrison, 1977; Coleman and Prior, 1988). Specialized tools have been developed for both sampling and testing these weak profile sediments and for quantitatively assessing slope stability during major storms (Suhayda et al., 1976; Prior et al., 1989; Hooper and Suhayda, 2005). Hurricane Ivan was a storm of sufficient magnitude to initiate large-scale sediment instabilities and sediment transport to upper continental slope water depths, leading to the Taylor Energy problems in MC 20.

Complexity of the MC 20 project site is clearly evident from observing the position of the lease block in the context of the large-scale sediment instability picture for the larger Mississippi River Delta offshore region (Figure 1). Details of this difficult regional geologic framework are apparent in the multibeam image of Figure 2. The well bay and toppled platform are in an area of overlapping mudflow lobes. Several large and relatively young lobes in this area have come to rest on the upper continental slope just seaward of the shelf-slope break. These lobes are composed of sediment evacuated from upslope sites and transported toward the shelf edge through narrow gullies. The rough bottom character of both the gullies and the recent depositional lobes, illustrated on high quality multibeam bathymetry data (Figure 2), reflect the presence of both discrete blocks of stratified sediment moving down the "mudflow conveyor belt" and/or pressure ridges created primarily at the distal ends of recent depositional lobes (Roberts, 1980; Prior and Coleman 1978; Bouma et al., 1991). The variability in surface and shallow subsurface character of mudflow deposits translates into differences in geotechnical properties (Roberts et al., 1980). The pressure ridges are most pronounced on the relatively steep distal ends of the lobes which downlap on older, but similar mudflow deposits. Individual lobe heights above the surrounding seafloor can be as much as 70-80 feet and slopes on the distal ends of these lobes or mudflow "noses" can be as much as 3-4%. Secondary failures commonly occur in association with these distal lobe settings. One of these lobe front failures is clearly displayed in the multibeam image of Figure 2. It is clear from inspection of Figure 2 that the multibeam bathymetry data indicate that the most recent mudflow lobes are prograding over similar features with surfaces smoothed by normal hemipelagic deposition and turbid flow "overrun" processes after the mudflow deposits are rather rapidly put in place. This lobe stacking suggests a complex variability in sediment properties both vertically and horizontally through these deposits.

Summary Comments: *Over the last four decades ME and FMMG have collected high resolution geophysical data (subbottom profiles, side-scan sonar swaths, and multibeam bathymetry) to improve the state of knowledge of seafloor instability features and their critical geologic components. Coupled with geotechnical data from in situ testing and borings FMMC personnel certainly have the background and experience base to understand the important geologic and geotechnical characteristics of mudflow systems and their subcomponents. The FMFR coverage of these details reflects a solid understanding of the detailed characteristics of mudflows and the properties of their sediments.*

Key Stratigraphic Components: Boring 2 Site

During Hurricane Ivan (September 2005), the seafloor failure that toppled the MC 20 production platform accreted the pre-Ivan seabed by 40-50 feet sediment. However, the Fugro-McClelland Final Report (FMFR) states that the slide plane at the platform site was 90 feet below the pre-Ivan seafloor elevation, suggesting that erosion of pre-Ivan deposits could have taken place to a depth of 90 feet for a net deposition of approximately 130 feet of mudflow sediment at the site. It is estimated that Hurricane Katrina in 2005 changed the seafloor at the well site by adding less than 10 feet of additional mudflow material. Currently, the well jacket is 150-155 feet below the post-Katrina seafloor.

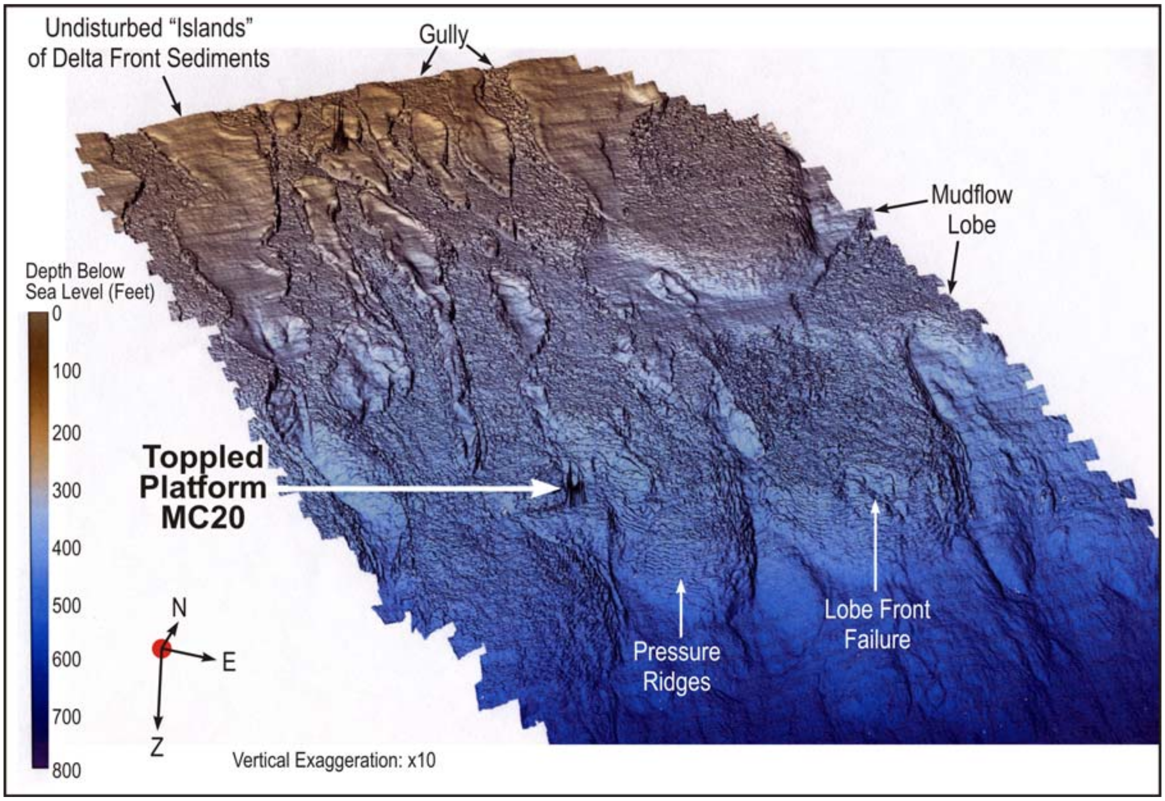

Figure 2. This multibeam bathymetry image of the project area around Mississippi Canyon Block 20 illustrates the extreme complexity of the surficial geology of this area. Smooth regions of seafloor updip of the MC 20 platform site are areas of undisturbed prodelta sediment resting on older unstable mudflow deposits. The elongated gullies between these islands of undisturbed seafloor are unstable sediment transport routes that deliver low shear strength mud to well-defined lobes. Through time these lobes are stacked and offset laterally (compensational stacking) as mudflow activity shifts from one location to another.

The location of Fugro-McClelland's Boring 2 is adjacent to the well site and therefore reflects current stratigraphic relationships at the former platform site. Three major stratigraphic units were recognized in the FMFR: (1) mudflow deposits, (2) debris flow deposits, and (3) pre-Holocene or pre-mudflow/debris flow shelf-slope transition deposits (identified as pre-mudflow deposits in the FMFR). Unlike mudflows or "fluidized sediment flows" in which the sediment is supported by escaping fluids, debris flows have large grains (rip-up clasts in the MC 20 case) that are supported by a matrix of fine-grained sediment and fluids (Middleton and Hampton, 1976). At the well site, the mudflow deposits and debris flow deposits are interpreted as being 125 feet thick and 25 feet thick respectively. However, an X-ray radiography study of selected subsamples for Boring 2 (FMFR) indicates that sediments interpreted as having been deposited by the mudflow process are at least 127 feet thick. Since the first debris flow deposit occurs at 130 feet below the mud line, the mudflow deposits could be as thick as 130 feet. These thick mudflow sediments display the following internal structures: (a) complete occlusion of any primary sedimentary structures by remolding during the transport process, (b) convective mixing of sediments during the transport process, (c) the inclusion of stratified "blocks", and (d) abundant evidence of gas expansion and migration (Roberts et al., 1976; Roberts 1980). Figure 3 shows these sediment characteristics using X-ray radiographs of selected samples from the lower part of the mudflow unit. Details of mudflow sediment characteristics will be discussed later in this report when the "weak zones" identified in the mudflow sequence (FMFR) are discussed.

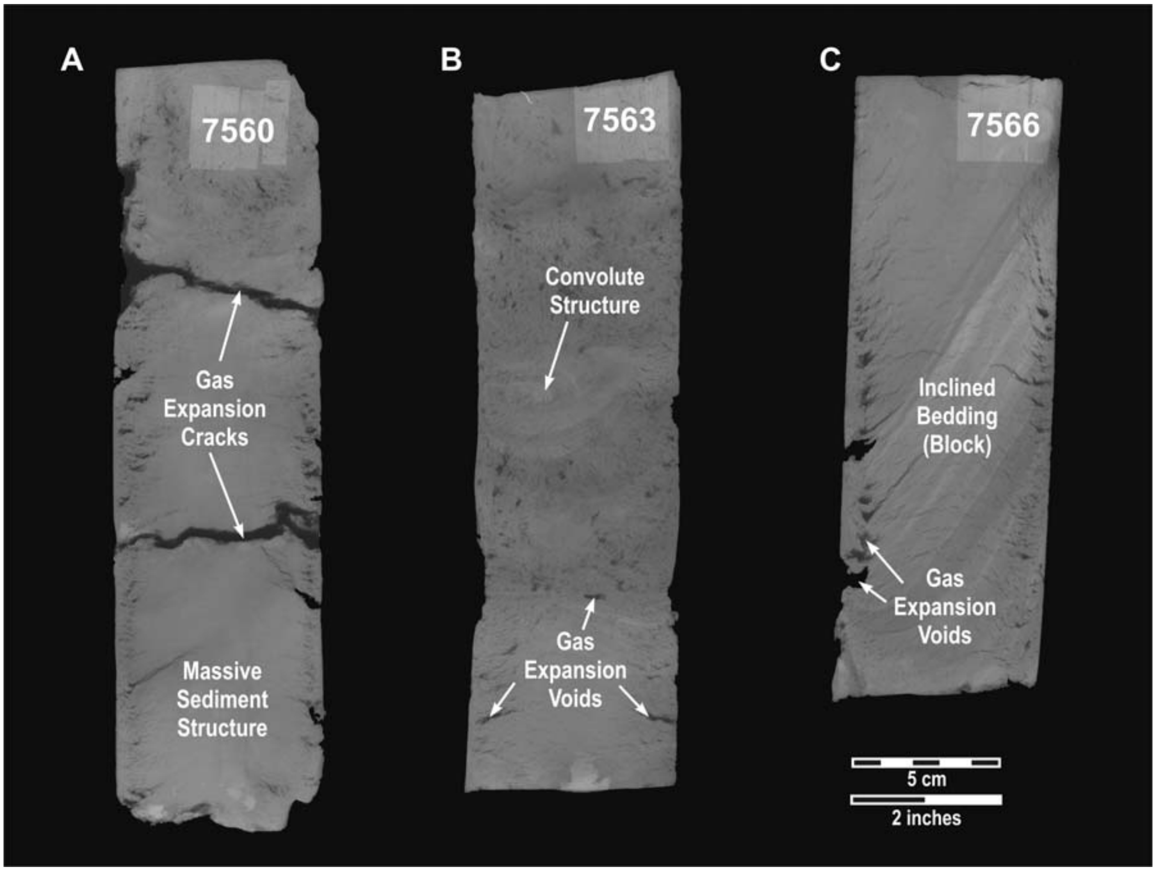

Figure 3. These X-ray radiographs from the lower part of the mudflow deposits at the Boring 2 site illustrate the interval characteristics of these types of low shear strength sediments. Three basic structures represent most mudflow deposits: (a) massive or completely remolded structure with gas expansion features (Sample 1B, 90-92 feet), (b) convolute structure resulting from turbulent mixing during the transport process (Sample 5A, 100-102 feet), and (c) inclined bedding which representing discrete and titled blocks in a matrix of more mixed and remolded sediments (Sample 7B, 105-107 feet).

The debris flow deposits, are identified as being 25 feet thick in the FMFR. Data from Boring 2 (X-ray radiographs) indicate that these deposits are found as deep as 178 feet below the mudline (Figure 4). Although it is true that the debris flow deposits do not occur continuously throughout the interval 130-178 feet (an interval of 48 feet), a thin debris flow deposit does occur at a stratigraphic depth of 178 feet and this horizon could function as a slide plane if deep sediment instability processes are reactivated at this site. Figure 4 illustrates the X-ray radiographs of the debris flow deposit at the 178 foot depth and the sediment beneath it which has internal structures and inclusions suggesting relatively slow deposition. It is clear that the debris flow clasts from all horizons are derived from erosion of preexisting outer shelf-slope deposits, especially in the interval 130-164 feet (interval thickness of 34 feet). Radiocarbon dating (AMS) in the middle of the 34 foot interval (samples at 141-141.8 feet) dated at 6,480 +/- 40 years BP. Clasts in this sample are from hemipelagic sediments deposited as the shelf was flooded following the latest Pleistocene glacial maximum and before the modern Mississippi River was delivering much sediment to the site. Dates of 21,390 +/- 80 years BP and 22,600 +/- 110 years BP acquired from nondeformed sediments that bracket the thin debris flow deposit at a depth of 178 feet suggest that the process or processes responsible for this thin unit are decoupled from debris flow deposits in the 130-164 foot interval. Therefore, mudflow sediments and associated underlying debris flow units that overlie older shelf-slope transition deposits end at a depth of 164 feet in Boring 2.

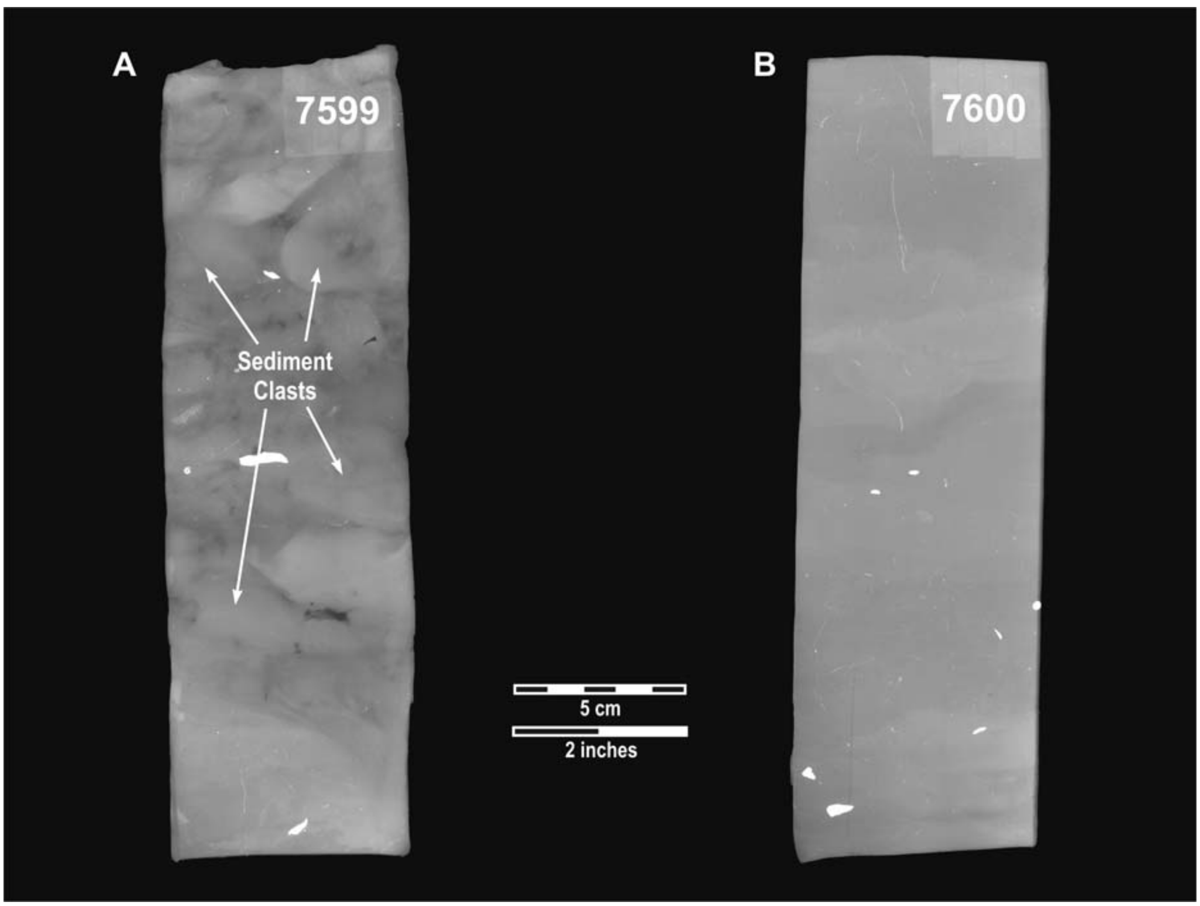

Figure 4. These two X-ray radiographs illustrate the (a) debris flow deposit (Sample 41A, 178-179 feet) and (b) the hemipelagic clay-rich sediments into which the debris flow eroded (Sample 41B, 179-180 feet). The debris flow deposit is bracketed by (Sample 37, 168-169.3 feet) and (Sample 45, 188-189.3 feet) which produced AMS dates of 21,390 +/- 80 yrs. BP and 22,600 +/- 110 yrs BP respectively.

Comparison of sedimentary units defined in Boring 2 to prominent reflection horizons in high resolution seismic profiles oriented along-shelf and cross-shelf through the boring site (using a speed of sound of 5500 fps) suggests that there may be more than 10+ foot discrepancy between unit boundaries in the boring and interpreted equivalent reflectors on the seismic profiles. The seismic horizons plot above the interpreted correlative boring horizons suggesting that a speed of sound slightly slower than 5500 fps is appropriate. The surface on which debris flow units downlap is interpreted to be at ~ 164 feet in Boring 2 (Figure 5). This surface is identified as Horizon 5 in the FMFR. Individual debris flow units are thin and below the resolution of even the high resolution subbottom profilers used in the MC 20 study. However, the prominent surface on which the interpreted debris flow reflectors successively downlap in a seaward direction represents the outer shelf-upper slope surface after sea level rose across the shelf and prior to significant sediment input from the Mississippi River. The seaward-stepping reflectors that downlap on Horizon 5 (FMFR) probably represent hemipelagic drapes deposited between periods of debris flow activity. Debris flow deposits separated by thin turbid flow and hemipelagic units are interpreted to be confined to the interval between 130-164 feet thick at the Boring 2 site. The Holocene date (6,480 +/- 40 yrs BP) acquired for a sample at 141 feet suggests incorporation of old hemipelagic sediment into younger sediments. The X-ray radiographs of the debris flow units (FMFR) clearly indicate that the rip-up clasts incorporated in these deposits are largely hemipelagic in origin. Debris flow deposits are difficult to date because of the mixing of calcareous microfossils of different ages. Even units above and below the debris flow deposits contain turbid flow deposits which can be erosive and thereby carry "old" calcareous microfossil tests. However, the sample at 141 feet is Holocene in age and the debris flow deposition probably was much younger than the date suggests. Since no dates were acquired on the debris flow at 164 feet, it is uncertain that this thin unit is stratigraphically above Horizon 5 and therefore part of the Holocene debris flow unit.

Thin debris flow unit at the 173 foot stratigraphic horizon indicate that this debris flow deposit is not associated with Holocene deposition. The data from Boring 2 show a slight reduction in shear strength in this interval. Sediment type and geotechnical data suggest that this horizon could function as a slide plane if a mudslide eroded through the uppermost shelf-slope transition deposits to near the 173 foot level.

Summary Comments: *The key stratigraphic components (mudflow deposits, thin debris flow deposits, and pre-mudflow/debris flow strata) are identified from Boring 2. The FMFR clearly distinguishes these stratigraphic units and the characteristics of each. However, the thickness of the unit representing combined debris flow deposits is underestimated by 9 feet (25 to 34 feet). Also, there is a mismatch between the seismic reflection events and boundaries for the stratigraphic components derived from Boring 2 data. The mismatch is in the range of 10+ feet. Velocity changes through mudflow and debris flow deposits and perhaps errors in determining the mudline at the boring site may be responsible. This correlation problem is not critical to the sediment excavation feasibility study, but it does indicate the difficulty of establishing clearly-defined stratigraphy within complex mudflow deposits, even with very good data sets.*

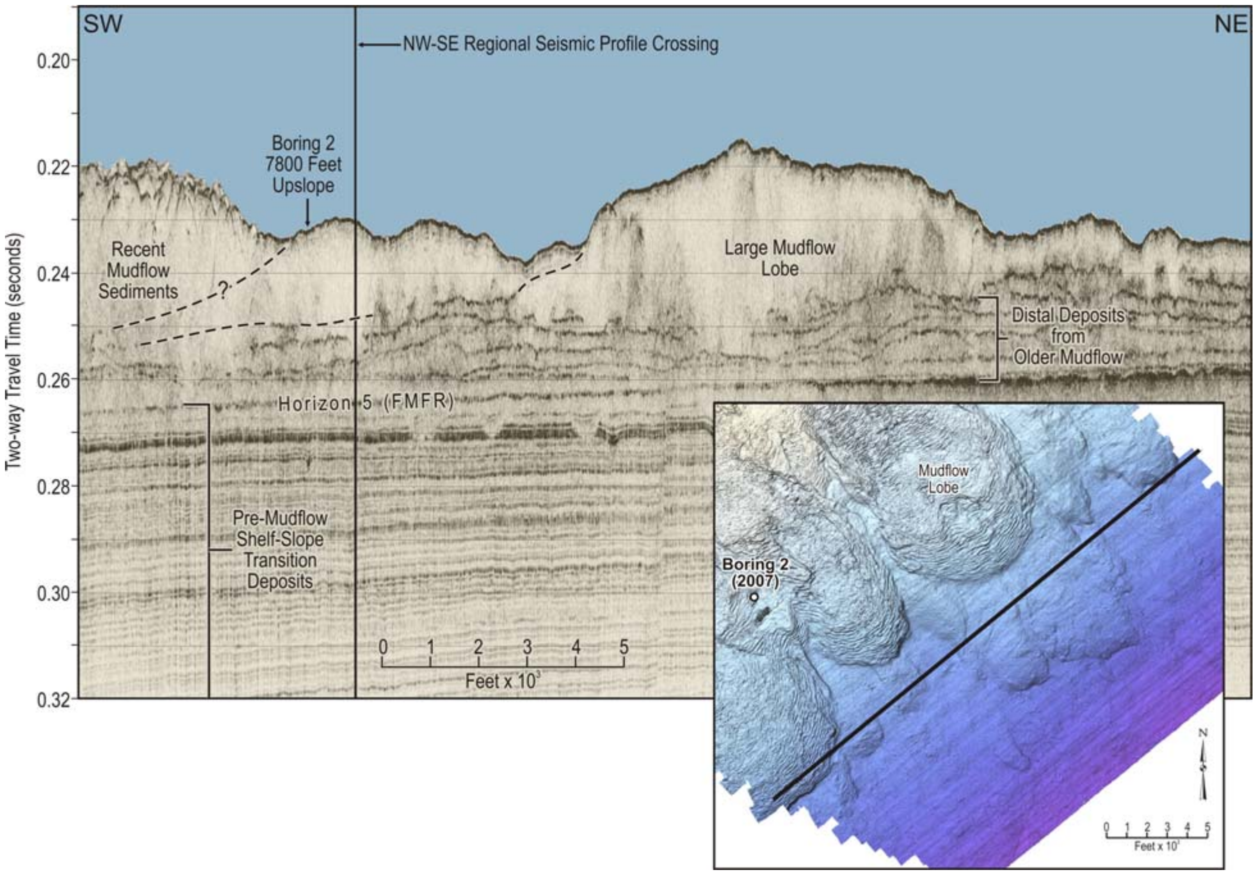

Figure 5. This figure presents a SW-NE oriented high resolution seismic profile that cuts across stacked and offset mudflow lobes seaward of the Taylor Energy Company platform site in MC 20. Note the lack of parallel or even subparallel stratigraphy in the mudflow deposits above Horizon 5 (FMFR). The inset map generated from multibeam bathymetry data shows the location of the seismic profile with respect to mudflow lobes and Boring 2. The crossing point for a NW-SE oriented seismic profile illustrated in Figure 6 is shown.

Mudflows Deposits: Stratigraphic Architecture, Sediment Characteristics, and Stability

In the previous section the major stratigraphic units as defined in the FMFR were discussed. Thickness of the acoustically opaque mudflow deposit was estimated to be 130 feet while the thickness of the debris flow unit was modified slightly to be thicker than reported in the FMFR. Data from the supporting X-ray radiography report (FMFR, Appendix C) support a thicker unit. The FMFR states that these deposits are approximately 20 feet thick. Analysis of selected samples by X-ray radiography indicates that this interval of thin, stacked debris flow deposits is approximately 34 feet thick. The ages are problematic, but on seismic profiles the debris flow deposits should fit between FMFR Horizons 4 and 5. However, as discussed in the previous section, there is a problem rectifying the seismic data with the sedimentary units determined from Boring 2.

The mudflow deposits generally appear acoustically opaque on high resolution seismic profiles while the thin debris flow deposits are subparallel and appear to downlap on Horizon 5 (FMFR). A lack of internal reflection horizons in the thick mudflow section is related both, (a) erosion of underlying sediments and the convective mixing process during transport and (b) the occurrence of bubble-phase gas scattered throughout the deposit (Figure 3). The transport process remolds the sediment and eliminates primary sedimentary structures. Gas bubbles cause diffusion of acoustic energy and consequently obscure reflection events. The clay to silty-clay sized sediments of mudflow lobes and gully deposits contain a high organic content primarily from degraded plant debris carried by the Mississippi River. Microbial degradation of the material leads to the production of biogenic methane which resides in the sediment both as bubble phase gas and gas dissolved in pore water (Whelan et al., 1975, 1976). Therefore, the stratigraphy architecture of mudflow deposits is difficult to determine using seismic profiling. However, "massive" appearing mudflow lobes may be the products of multiple depositional events. Even though mudflows are erosive and may partially or totally eliminate previous mudflow deposits in their paths, most thick and generally acoustically opaque mudflow units probably represent stacked depositional lobes. When discontinuous but definable reflectors in otherwise opaque-appearing mudflow deposits are correlated to sediment samples from borings, the reflectors commonly represent thin, stratified "interlobe deposits" (Roberts, 1980). These are relatively slowly deposited sediments on the surface of a rapidly deposited mudflow lobe after mudflow deposition has stopped. The next mudflow may erode part of the "interlobe deposits," but remnants are left. These remnants provide the acoustic impedance contrast within mudflow lobe deposits to produce a discontinuous reflection event. Large mudflow lobes like those in the MC 20 area commonly exhibit fragmented internal reflection horizons that suggest deposition from multiple mudflow events. On both 2D-multichannel seismic profiles and high resolution subbottom data there are "ghost reflectors" internal to the mudflow unit (mudline to 130 feet in Boring 2) within the MC 20 study area.

Geotechnical data derived from subsamples of borings scattered throughout the project site indicate that the mudflow sediment is of low shear strength (< 0.4 ksf) and underconsolidated throughout to the contact with underlying debris flow deposits at a depth of approximately 130 feet in Boring 2. An interesting abrupt increase in water content (60% to ~ 80%) and liquid limit occurs just above the mudflow-debris flow boundary. These changes were accompanied by a reduction in shear strength. In Boring 8, a similar decrease is observed at a subsurface depth of about 150 feet. These two subsurface horizons characterized by drops in shear strength are roughly correlated to Horizons 4 and 5 at the Boring 8 site in the FMFR. Also, there are two weak layers that are evident in the shear strength profiles of the borings from the project area. These zones of weakness occur in the stratigraphic intervals 80-100 feet and 115-130 feet. Selected samples from Boring 2 that were studied by X-ray radiography indicate that core samples from the upper interval tend to be thoroughly remolded and display abundant evidence of gas expansion and migration. Directly below the 100 foot level the internal character of the sediments suggests the existence of titled blocks of relatively intact sediment incorporated in the remolded mudflow unit. Within the lower interval, sediments are again thoroughly remolded and contain evidence of sedimentary gas. Although there are no consistent reflection horizons that correspond to these depths in the high resolution seismic profiles presented in the FMFR or in my personal evaluation of the seismic database archived at the Fugro-McClelland office in Houston, the following statement was made in Fugro-McClelland

Report No. 0201-6235-1 regarding the deeper weak layer: "There is no seismic or other evidence to suggest that this weak zone is localized. Rather, these horizons represent geologic surfaces that are interpreted to extend over large areas, and (weak) strength profiles are likely to also be similar over a large area." In my opinion, it is not probable that surfaces or units (uniform stratigraphy) extend over large distances in mudflow terrain. The morphology and internal stratigraphy of thick mudflow lobes suggests that they are features accreted by multiple depositional events. The multibeam bathymetry data emphasize that multiple lobes stacked and offset from one another characterize mudflow deposition in the project area. Figure 5, an along-shelf high resolution seismic profile across multiple mudflow lobes, clearly illustrates the compensational stacking style of these depositional features.

The Figure 5 profile was selected for illustration purposes because it clearly shows the stacking pattern of both the distal "toes" of older mudflow deposits and the thicker acoustically opaque younger lobes. The rough-appearing surface of the youngest lobe at the SW side of the profile has parabolic acoustic returns derived from irregularities (blocks and pressure ridges) at the surface of the lobe. Post-mudflow deposition has filled in irregularities on lobes to the NE. Beneath the thick acoustically opaque mudflow deposits are the offset and stacked thinner units which represent the distal "toes" of mudflow deposits separated by strong reflectors. The acoustic impedance difference between the mudflow sediment and these thin high amplitude reflectors is the difference in properties between remolded delta-front mud and thin silt-rich turbid flow deposits mixed with hemipelagic sediments containing abundant calcareous microfossil tests.

The point of illustrating the compensational stacking patterns of mudflow lobes and the variations in sediment types is to emphasize the potential lateral and vertical variability in sediment types and their corresponding geotechnical properties. Whatever else is true of composite mudflow deposits that dominate the outer shelf and upper slope opposite the modern Mississippi River Delta, they do not describe "layer cake" geology. These are complex interfingering deposits that range from highly gassy and thoroughly remolded clays and silty-clays to true hemipelagic sediments with a substantial content of shell material (mostly from pelagic foraminifera) that separate mudflow units (Roberts, 1980). Occasionally, localized sand-sized sediments are encountered, but not often. The stratigraphic architecture illustrated in the seismic profile of Figure 5 strongly suggests that assumptions of lateral continuity of geotechnical properties cannot be supported. Specifically, the "weak layers" determined from Borings 2 and 8 cannot be projected as "geologic events" that are present as consistent layers throughout the sediment excavation project area. Clearly, weak sediments are characteristic of both mudflow lobe and gully deposits. However, the stacking patterns of sediments of different engineering properties affects slope stability. The natural lateral variability of depositional units and their associated geotechnical properties has profound implications on modeling the stability of a large-scale sediment excavation, the subject of the Fugro-McClelland feasibility study.

Summary Comments: *The array of soil borings acquired in support of activities in the MC 20 area cover a relatively small geographic area when compared to the calculated dimensions of an excavation using even a marginal factor of safety (FOS = 1). These soil borings provide the geotechnical data on which the excavation design is based. Weak zones found in Borings 2 and 8 are propagated over large distances as part of the excavation design* *criteria. Fundamental patterns of erosion, deposition, and mudflow lobe stacking in the project area eliminate the probability that sedimentary units and their associated geotechnical properties can be projected very far laterally in this compensationally stacked mudflow lobe setting where the stratigraphic architecture is extremely complex. In my view, this depositional environment does not lend itself to a simple "layer cake" modeling of depositional units and their properties. Even though the FMFR acknowledges the difficulties of the MC 20 geologic setting, nested instabilities in the natural system related to mudflow stacking patterns and intersecting gullies may introduce complexities beyond the basic assumptions used for the engineering projects.*

Sediment Excavation and Slope Stability

At the well bay site, the 28 conductors (30 inch diameter) are currently buried by approximately 150 feet of sediment. The FMFR focuses on an engineering design to remove sediment at the well bay site to a depth of 180 feet below the current mudline. The purpose of the excavation is to expose the conductors so they can be plugged and abandoned in accordance with MMS regulations.

For the excavation design (slope stability analyses) the upper and lower bound profiles for soil strength and weight were used in order to account for the variability of these properties at the project site. As discussed in the previous section of this report, because of the way mudflow deposits are stacked at the shelf slope transition area it is difficult to imagine that sediments sampled by a single boring or a tight cluster of borings could capture the variability of soil properties encountered by an excavation (designed for a FOS=1.0) that extends 17,000 feet upslope and 2,700 feet downslope from the well bay. A complex of overlapped and stacked mudflow lobes has numerous depositional and erosional contacts as well as sediment types (remolded sediments, blocks, turbidites, debris flow units, and hemipelagic deposits). The complex stratigraphic architecture of these deposits houses a wide range of soil properties that probably makes simple slope stability models, such as the parallel stratigraphy model used in the FMFR, a significant over simplification of the real world. Therefore, such a model may not be a reliable predictor of slope stability over large areas. Figure 5 emphasizes the complexity of the deposits throughout the project area. This figure was discussed previously as an example of the complicated stratigraphic framework for mudflow and possibly debris flow deposits resting on the outer shelf and upper continental slope. Figure 6 illustrates the NW-SE cross-shelf stratigraphy on a seismic profile through the Borings 2 site. The multibeam bathymetry image (inset for Figure 6) illustrates the location of the seismic profile with regard to mudflow lobes, gullies, and islands of undisturbed delta-front sediment. Superimposed on this seismic line is the excavation profile using the lower-bound slope stability case (FOS=1.0). Under these marginally stable conditions, the excavation area covers 9 mi^2 and the dredge volume is $60x10^6$ yd^3.

Using the lower bound case for the slope stability analysis with the marginally stable slope FOS = 1.0, the slope of the excavation wall north of the well bay was determined to be 1.3º. This value is almost three times the 0.5º slope of the natural delta-front where numerous slope failures occur. As both Figures 5 and 6 clearly demonstrate, the excavation would extend beyond the outer shelf-upper slope mudflow lobes into mid-shelf mudflow gullies and islands of undisturbed delta-front sediment. Upslope projections of the excavation (FMFR) indicate that 5 mudflow gullies and 4 islands of undisturbed sediment would be encountered. The gullies are transport routes that function as the sediment feeder systems for the mudflow lobes at the MC 20 site and adjacent to it. Gullies contain unstable sediment and a seabed profile down any given gully will describe a series of breaks in slope that represent potential points of instability along the transport route (Roberts et al., 1980). No geotechnical data were acquired from the gullies. However, it is highly probable that sediments of equivalent or even lower shear strength than the lower bound value from the mudflow in the well bay area could be encountered in the gullies.

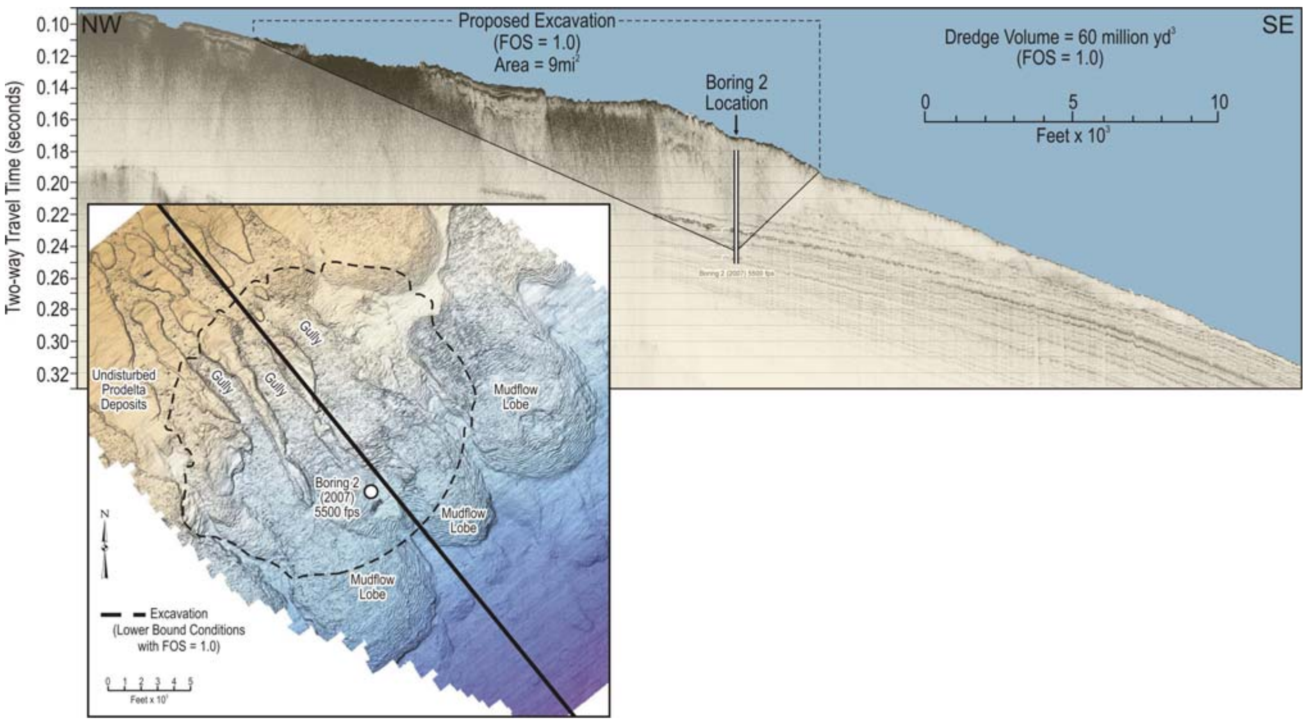

Figure 6. Superimposed on the NW-SE oriented high resolution seismic profile is the projected excavation feature centered at the well bay and extending upslope through mudflow lobes, gullies, and relatively undisturbed "slabs" of delta-front sediment. The inset image generated from multibeam bathymetry data shows the position of the seismic line with respect to geologic features of the project area. The general outline of the excavation (FOS = 1.0) is superimposed on the image.

The undisturbed islands of seafloor appear "smooth" on the multibeam image of Figure 2 and stratified in profile view (Figure 6). These areas are composed of relatively undisturbed delta-front sediments that have been deposited over older mudflow lobes and gullies. That is, they are thin wedges of prodelta clay and silty clay deposited over weak older remolded deposits as the entire delta-front progrades seaward. Since the modern Mississippi River Delta is only 600-800 years old (Coleman et al., 1980), all of the sediments in the project area above Horizon 5 (FMFR) are probably less than 400 years old. The fact that all these sediments are young suggests that they are all highly underconsolidated, exhibit excess pore pressures, and consequently have low shear strength profiles because of rapid deposition and the fact that they are generally fine grained (Coleman et al., 1980; Roberts et al., 1980). Removing these sediments by dredging to a design slope of 1.3º is probably not feasible without continued wall failures. Undercutting the slabs of "undisturbed" sediments would no doubt initiate retrogressive slumping as occurs on the upper delta-front at the heads of gully systems.

Design estimates of the excavation area and sediment volume were also made for a higher and more stable FOS = 1.3. Using this FOS and the lower-bound soil parameters, the excavation area would be 80 mi^2 and the dredge volume is calculated to be $50x10^9$ yds^3. A map projection of the excavation outline from the FMFR using the FOS = 1.3 shows the northern limits of the excavation intersecting subaerial components of the Modern Mississippi River Delta. A FOS = 1.3 is not even the initial target of 1.5. An excavation necessary to attain this target FOS =1.5 is certainly not feasible in terms of scale of operation. Even if the excavation scaled to a FOS = 1.0 were acceptable, the disposal of $60x10^6$ yd^3 of sediment in an environmentally acceptable way would be an enormous challenge. The shelf-edge and upper continental slope are sites of sensitive benthic communities such as hard and soft corals as well as chemosynthetic communities. Corals and chemosynthetic communities are protected by federal law. It is the responsibility of the Minerals Management Service to protect these sensitive biologic communities from human activities in the marine realm. Even though FM was not tasked to address the sediment disposal problem, handling this problem in an environmentally acceptable way is key to the success of any excavation project.

Summary Comments: *Modeling sediment stability and designing a sediment excavation feature centered at the well bay site with a target depth of 180 feet is not a simple task in mudflow sediments. Any model is a simplification of real world conditions using parameters judged to be most important for making reliable predictions of real world response from model output. Critical soil parameters used for the modeling were derived from borings clustered near the well bay area which sampled mudflow lobe and underlying debris flow deposits. Weak layers found in these deposits were projected throughout the project area using a slope stability model that incorporates horizontal stratigraphy. Model results using a marginally stable FOS = 1.0 define a large excavation that extends upslope into mudflow gullies and thin islands of "undisturbed" sediment deposited on top of old remolded mudflow deposits. Considering the extremely complex stratigraphy of the mid-shelf to upper slope deposits resting on older subparallel shelf-slope units (below Horizon 5, FMFR), it is doubtful that the "layer cake" model adequately predicts the nested instabilities in the young gully-mudflow-debris flow section. It follows that the model probably doesn't predict the stability of the excavation feature very well. In my opinion, results of the lower bound model using a FOS = 1.0 must be viewed as extremely optimistic. When considering the complex stratigraphic architecture and factoring in the effects of cyclic loading by ocean waves generated by cold front passages and possible hurricanes, it seems doubtful that wall integrity of the excavation could be maintained for a period sufficient to plug and abandon the 28 wells (estimated to be 400 + days, personal communication, Casey Geohegan of Taylor Energy Company, Inc).*

In addition, the volume of sediment to be extracted from the excavation features, even in the most conservative case driven by a FOS = 1.0, is enormous. Disposal of this sediment is an environmental challenge that has direct impact on the feasibility of any large-scale excavation project.

Maintaining Wall Stability on the Excavation Under Storm Wave Forcing

In the previous two sections of this report, I have attempted to point out that complexities in mudflow stratigraphy and corresponding sediment properties create nested instabilities in the sediment column that seem not to be accounted for by the slope stability model. In addition to the slope instabilities that arise from gravity acting on complex stratigraphic contacts and sediments with different properties, slope failures are frequently forced by oceanic processes, particularly wave forcing.

Waves generated by strong winter storms and hurricanes cyclically load bottom sediments with pressures related to wave height, wavelength, and water depth. Storm waves represent many different wave heights and periods. The greatest bottom pressures are caused by the waves of greatest height and longest periods. Waves associated with Hurricane Ivan, for example, were estimated to have reached heights as much as 90 feet (Hooper and Suhayda, 2005). From the Mississippi Delta distributaries across the entire delta-front and onto the upper continental slope, thick accumulations of weak Mississippi River clay and silty-clay are impacted when storm waves transit the northern Gulf. With the passage of a single storm wave the bottom is depressed under the peak and moves upward under the trough. The amount of movement is controlled by geotechnical properties of the sediment (e.g. shear strength, viscosity etc.) and wave characteristics. Zhang and Suhayda (1994) point out that sediment properties don't remain constant during cyclic wave loading but gradually degrade over time. A steady degradation of engineering properties increases sediment motion during the passage of waves and at the same time decreases critical properties like shear strength. In addition, to changing engineering properties, the cyclic pressure changes associated with wave crests and troughs can cause dissolved gas (mostly biogenic methane) in the pore waters to shift state to bubble phase. Both degradation of engineering properties and a transformation from dissolved-to-bubble phase gas under cyclic loading of the seabed by storm waves can result in slope failures when dealing with fine-grained and underconsolidated delta-front mudflow deposits.

The FMFR did not deal with the affects of wave loading on the excavation modeled with both a FOS of 1.0 or 1.3. A period of 400 + days of exposure was estimated for divers to prepare, plug, and abandon the 28 wells at the MC 20 site (personal communication, Mr. Casey Geohagan of Taylor Energy Company, Inc.). For such a time-frame storms from at least one hurricane season and perhaps two could impact the project area, depending on start time and number of "weather days" for dive operations. Since the scale of the excavation feature using a FOS = 1.3 is probably beyond technical feasibility, the marginal stability case of FOS = 1.0 creates slopes of 1.3º to the NW of the well bay and 2.5º SE of the well bay. If hurricane waves impact the excavation during the period when wells are being prepared and plugged, it is highly probable that the excavation walls will fail and cover the well site once again with thick mudflow deposits.

Summary Comments: *Cyclic loading of the delta-front by hurricane waves has been documented as the cause of slope failures leading to broken pipelines and toppled production platforms (Bea, 1971; Bea et a., 1975; Sterling and Strohbeck, 1975). The FMFR provided the feasibility of producing a sediment excavation feature for exposing the well bay at the Taylor*

Energy MC 20 site. However, the wall stability of the two cases for the excavation (FOS = 1.0 and 1.3) was not modeled under cyclic loading by hurricane waves. Since the time required for divers to prepare and plug the 28 wells is estimated at 400 + days, it is possible that a hurricane may impact the Gulf during the period. Wave effects on wall stability of the excavation were not included in the investigation. The FMFR acknowledges the importance of storm waves on slope stability and that future sediment mass-transport events may be triggered by hurricane waves which may funnel sediment to the MC 20 area and beyond. However, modeling of wave effects on slope stability was not included in the FMFR.

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APPENDIX B

Reliability Based Factors-of-Safety

for Excavation Project, Block 20,

Mississippi Canyon Area, Gulf of Mexico

Report to

Taylor Energy Company

New Orleans, Louisiana

by

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March 2008

Reliability Based Factors-of-Safety

for Excavation Project, Block 20, Mississippi Canyon Area, Gulf of Mexico

Factors of Safety

The Factor-of-Safety (FS) is intended to be a safeguard against uncertainties associated with the maximum demands (S) an engineered system will experience and the abilities of the system withstand those demands – its capacities (R).

Because we are not certain about either the maximum demands nor the capacities, we want the system to be 'safe' and hence the desire to separate S and R with a FS. Generally, the FS is defined as the ratio of a specific R (R_d) to a specific S (S_d); often described as the ratio of the design capacity to the design demand. The common interpretation is that if the FS is greater than unity, then the system is ‘safe’ and that if the FS is less than unity the system will have failed and is ‘unsafe.’

The primary method used to derive the FS has been experience. A system is designed for a certain purpose with a FS (implicit or explicit), it is built and put into operation. If that experience is good during the useful life of the system, then we conclude that the FS is satisfactory. If that experience is not good, then we conclude that the FS is not satisfactory, and the system is revised to have a larger FS.

The primary problems with the experience based approach is that it requires a long period of time to develop the necessary experience and failures can be very painful. So, we would like to have an approach that does not require such a long period of time and can hopefully avoid the pain of failures.

In the 1930's work began to develop FS based on analytical processes (Bolitin in Russia). By the 1950's this analytical process had been developed and was being used in design of some structures and engineered systems. The technology was identified as reliability based design criteria.

Since that time, the technology associated with determining FS has experienced dramatic growth. This approach is being widely used in a wide variety of engineered systems. But, for many engineers, this technology is not familiar. These engineers just want to have a FS that can be used to design a given system. These engineers know that the design process is fundamentally deterministic - the system must be configured in a certain way for certain conditions. So, the background for the FS generally remains behind the curtain of experience (frequently documented in design codes and guidelines) and that of reliability based design criteria.

Those who have practiced and applied reliability based design criteria understand that the technology is not magic. The technology provides a formal means to examine some of the important elements that can affect the acceptable and desirable performance of a system and provide information for decision makers to develop judgments concerning an appropriate FS.

Another element of understanding has developed. The term FS is appropriate when one is considering an element that comprises a system. Another term is appropriate when one is considering the assembly of elements - the 'system'. For structural systems comprised of assemblies of elements, the term Reserve Strength Ratio (RSR) has been coined to express the ratio of the ultimate capacity of the system to the design demand imposed on the system. The difference between the RSR and the FS is the difference in how the assembly of elements perform compared with that of the individual elements.

The traditional format used in the United States for design guidelines has been the Working Stress Design (WSD) format. This format generally utilizes a nominal ‘static’ loading to define the serviceability response characteristics and strength of the structure. Linear elastic analyses are used to describe the structure response characteristics for the given nominal design loadings. Based on characterization of the demands and capacities as being independent and Lognormally distributed, the traditional design factor-of-safety (FS_d) in WSD can be developed as:

$$FS_d = R_d / S_d = Fe_{50}\ (B_{S50} / B_{R50})\ \exp\left[(\ \beta\ \sigma_{lnSR}) - (\ 2.33\ \sigma_{lnS})\right]$$

Fe_{50} is a factor (median loading effect) that incorporates the interactive effects of dynamic - transient loadings and the nonlinear behavior of the system. B_{S50} is the median Bias (true value / nominal value) in the maximum demand (loading), B_{R50} is the median Bias in the capacity of the element. β is the ‘target’ (desired, acceptable) annual Safety Index (a measure of the desired reliability). σ_{lnSR} is the total uncertainty in the demands and capacities (standard deviation of the logarithms). σ_{lnS} is the uncertainty in the annual expected maximum loadings. σ_{lnS} is the standard deviation of the logarithms in the expected annual maximum loadings imposed on and induced in the structure system. This expression for FS_d is based on the premise that ‘100-year’ demand (99^{th} percentile of annual maximum demands, +2.33 standard deviations greater than the mean annual maximum demand) is used as the design demand (S_d).
The reliability (Ps) of the system can be expressed as the likelihood that the system R will equal or exceed the system S:

$$Ps = P[R \geq S]$$

The probability of failure (Pf) is the compliment of the system reliability:

$$Pf = 1 - Ps$$

The Safety Index is a non-dimensional ‘proxy’ measure of the FS and is related to the Pf as follows:

$$Pf = 1 - \Phi\ (\beta) \cong 0.475\ \exp\text{-}(\beta)^{1.6} \cong\cong 10^{-\beta}$$

where Φ is the Standard Cumulative Normal variate.

The total uncertainty in the logarithms of S and R for independent demands and capacities can be determined from:

$$\sigma^2_{\ln RS} = \sigma^2_{\ln R} + \sigma^2_{\ln S}$$

where σ_{lnS} is the uncertainty (standard deviation of the logarithms) in the annual maximum demands and σ_{lnR} is the uncertainty in the capacities of the elements. If the demands and capacities are not independent and are correlated (reflected in the correlation coefficient ρ_{RS}):

$$\sigma^2_{\ln RS} = \sigma^2_{\ln R} + \sigma^2_{\ln S} - 2\rho\sigma_{\ln R}\sigma_{\ln S}$$

The transient / dynamic loading - nonlinear performance factor, Fe, is dependent on the ductility (strain - deformation capacity), residual strength (load - stress capacity beyond yield), and hysteretic (cyclic load - deformation - damping behavior) characteristics of the structure. It is also dependent on the transient / dynamic loading characteristics including the duration of the imposed or induced loadings, the periodicity, and the force-time characteristics of the loadings. The important thing is to recognize that there can be very important differences between loadings or demands that are determined on the basis of static analyses of idealized elastic systems subject to non-time varying demands or loadings. Dynamics (time changing demands) acting on nonlinear inelastic systems (operating at or close to their Ultimate Limit States) can have very important effects on the FS or RSR.

Generally, the 'true' ultimate capacity of the element (Ru) is not used in the design process, and another 'nominal' or design capacity (R_D) is used. The capacity bias is introduced to recognize this difference:

$$B_R = Ru / R_D$$

In a similar manner, the loading bias is introduced to recognize the difference between the design or nominal demand (S_D) and the 'true' maximum demand (S_M):

$$B_S = S_M / S_D$$

The results of the foregoing developments are summarized in Figure 1 for Biases in the demands and capacities of unity and for a 100-year design loading condition. In developing these results it has been assumed that the total uncertainty is equal to the loading uncertainty. This is equivalent to assuming that the resistance or capacity uncertainty is negligible compared with the demand uncertainties (generally, this is a very good approximation). Some interesting trends are indicated in Figure 1. For low Safety Indices ($\beta \leq 2.5$), the FS and RSR are about unity and there is little significant variation in these parameters with uncertainty in the demands and capacities. For high Safety Indices ($\beta \geq 3.5$), the FS and RSR are very sensitive to the uncertainties. For low uncertainties ($\sigma \leq 0.2$), there is little change in the FS and RSR as a function of the target reliability expressed in the Safety Indices.

This development shows that the FS can be evaluated from evaluations that quantify the following parameters:

- the required level of safety reflected in the Safety Index β
- the uncertainties in the demands and capacities reflected in $\sigma_{\ln R}$ and $\sigma_{\ln S}$ and if there is correlation between the demands and capacities in the demand-capacity correlation coefficient ρ_{RS},
- the biases incorporated into the design demands and capacities, B_{S50}, B_{R50}, and
- the non-static (dynamic) effects not included in the design demands and capacities, Fe_{50}.

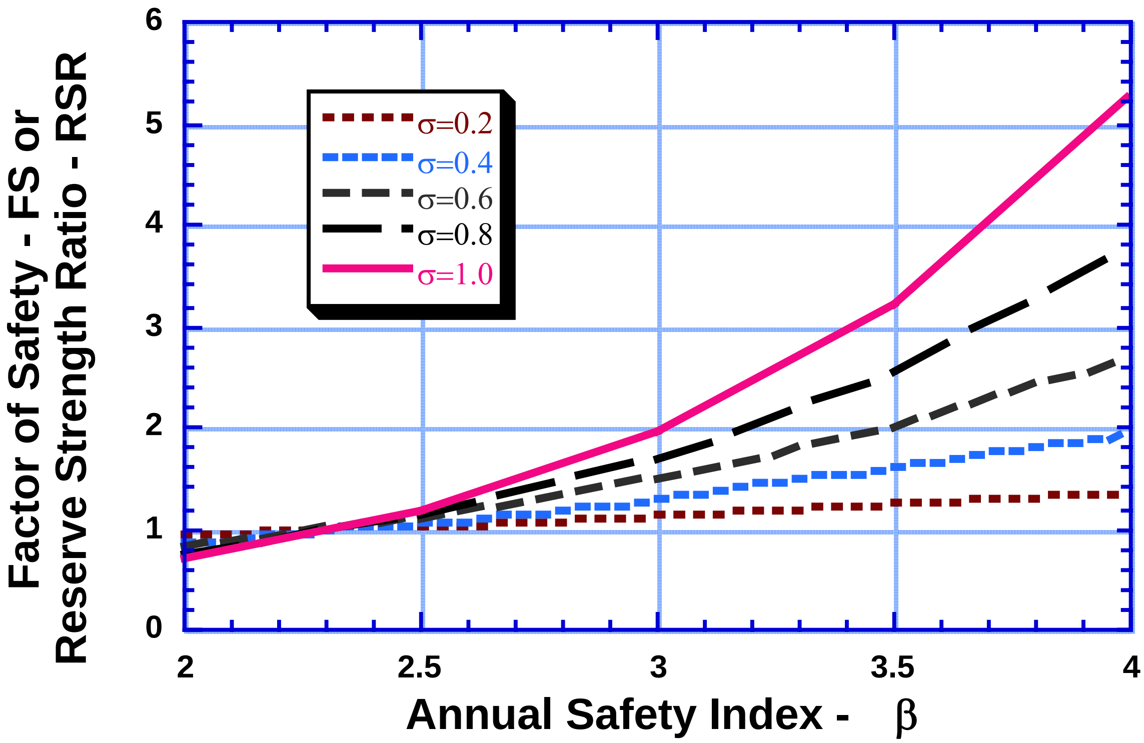

Figure 1- Element factor of safety or system Reserve Strength Ratio as function of total uncertainties in maximum loadings and capacities and biases of unity

It is important to note that the results summarized in Figure 1 are based on 'continuous' demand and capacity variables. If there are effective 'truncations' (physical limitations) in either the demand or capacity variables or both, then special treatment of the continuous distributions must be introduced. Important truncations in demand variables often come because it is not possible for the demands to exceed certain physical limits (e.g. wave heights in shallow water). Important truncations can also be introduced through 'proof testing'. Such testing establishes that it is not possible for the distributions to extend above or below values equal to the proof testing demands or capacities.

One method that can be used to recognize truncations is to treat the 'proof loading' (truncating the capacity distribution at the proof loading) as defining the capacity as a deterministic value equal to that used in the analyses (e.g. median value) and incorporating the total uncertainties in the demand variable. Thus, the probability of failure becomes the probability that the demand value is equal to or greater than the defined capacity variable. If the demand and capacity variables were defined as the median values and the distribution of the capacity were Lognormal, the Factor of Safety could be determined as follows:

$$FS = \exp(\beta\, \sigma_{\ln S})$$

Target Reliability

Target reliabilities are those that will be used to engineer and manage an engineered system during its life. There are a variety of complimentary ways to help arrive at decisions concerning these target reliabilities. Such decisions must be made by those who bear the primary responsibilities for the activities associated with the systems and for the consequences associated with those activities. The system engineers can provide information and insights concerning the tradeoffs between reliability and ‘costs’. The system managers (corporate and regulatory) must provide the decisions regarding what target reliabilities will be used to engineer the facilities.

Economics – Utility Approaches

An economics based approach to develop target reliability information for the quality attribute of ‘safety’ can be developed as follows. Let the expected probability of failure to achieve adequate safety be $\overline{Pfj} = \overline{Pf2}$. This probability will be expressed on an annual basis (probability of failure per year of exposure).

Let the expected operating costs associated with the inadequate safety be $\overline{Cf2}$. Then $\overline{Rf2} = \overline{Pf2} \bullet \overline{CF2}$. Let the expected initial risks associated with varying Pf2 be a linear function of the logarithm of Pf2 that has a slope ΔCi (the cost required to change Pf2 by a factor of 10). The equation for the total risk (initial plus operating) could be differentiated and equated to zero to determine the minimum total risk and thus define the Pf2 that would develop the minimum total risk:

$$Pf2o = 0.4348 / Rc\ (pvf)$$

Rc is a ratio of the costs associated with the safety failure (CF2) to the cost required to reduce the Pf2 by a factor of 10 (ΔCi). This a non-dimensional measure of the initial and operating costs. The pvf is a Present Value Function that is used to bring to present value terms the future costs associated with the compromises in safety. In this example, the pvf will be based on replacement of the system in the case of the compromises in safety and a continuous discounting function:

$$pvf = [1-(1+r)^{-L}]/r$$

where r is the net discount rate (investment rate minus inflation rate) and L is the life or exposure of the system in years. For long life systems ($L \geq 20$ years), $pvf \approx r^{-1}$. For short life systems ($L \leq 5$ years), pvf = L. The product of Rc times pvf is an expression of the ‘effective life’ of the system. Another ‘boundary’ on the safety risk could be defined by equating the slope of the total cost expression to unity. It is at this point that there is a rapid rise in the total expected cost with the change in Pf2. In this case the expression becomes Pf2m = 2 Pf2o, or the ‘marginal’ Pf is twice the ‘optimum’ Pf. The results are summarized in Figure 2.

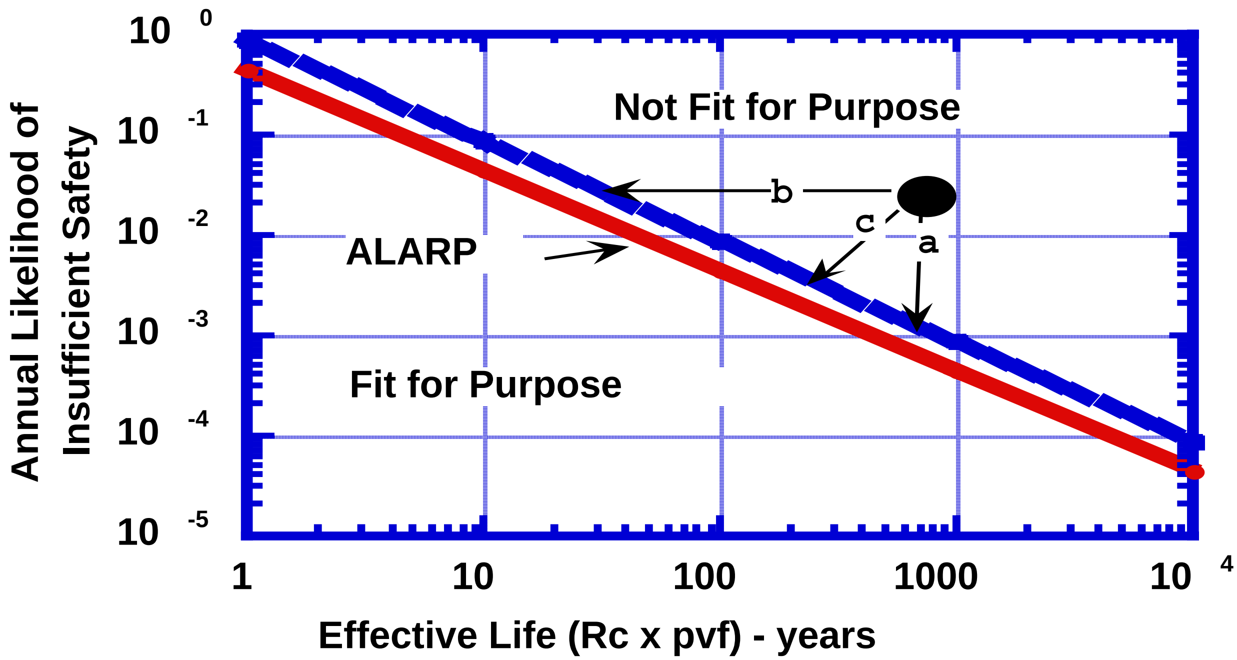

Figure2– Fitness for purpose reliability evaluation guideline

The line indicated as ALARP is the same as determined based on Pf2o. ALARP is taken to be 'As Low As Reasonably Practicable'. The dashed line indicates the marginal Pf. The combination of Pf and the measure of the initial and operating costs that lies above the dashed line is indicated as 'not fit for purpose'. The combination of Pf and the measure of initial and operating costs that lies below the solid line is indicated as 'fit for purpose'. The area between the two lines can be interpreted as the area that is subject to economic cost-benefit analyses to define an acceptable combination of likelihoods of failure and the consequences associated with the failure.

In the economics or utility based approach considerations of the broad category of potential 'consequences' associated with a failure is important. It is at this point that the very sensitive and difficult issues associated with potential environmental impacts must be addressed – including potential human impacts (injuries, fatalities). There are two basic approaches to address such impacts. The first is to address the potential impacts as a separate issue, not integrating the potential environmental injury (including human life) impacts with the other potential impacts (e.g. property losses, production and productivity losses). The second is to address the potential environmental impacts by integrating them with the same metrics to evaluate the other potential consequences.

It is at this point that the question is often raised: what is the value of a human life that should be included in the economics based approach? The reasonable way to pose the question is how much should be invested to save a human life in association with the proposed system operations? Terms like ICAF (Incremental Cost of Averting a Fatality) have been developed to help answer such questions. Analyses of the ICAF implicitly integrated into current design guidelines for natural hazards (e.g. storms, earthquakes, North American, European) indicates ICAFs in the range of U.S.$1 to $10 millions (2000). Recent studies of societal ICAFs indicate values in the range of U.S. $1 millions for developed countries like the U.S. and Norway. For developing countries like Mexico, Brazil, China, and India the ICAFs are in the range of U.S. \$0.1 to 0.3 millions.

It must be remembered that the objective of the entire Target Reliability process is an attempt to identify the 'best' alternative for design and operation of a system. 'Best' means to identify that alternative (or alternatives) that can provide the best chance to realize a system that has desirable and acceptable quality and reliability during its life cycle. Once such a system has been identified (together with the associated risk assessment and management processes), then the objective shifts to implementing the risk assessment and management process during the system life cycle that can result in a system with 'zero failures' – the objective is desirable and acceptable quality in the context of resources that should be expended to achieve such goals.

Historic 'Actuarial Data' Based Approaches

A second approach is based on experience with engineered systems in which actuarial or historic data is used to identify historic precedents associated with other engineered systems. The premise of this approach is that societies and the organizations that are parts of these societies through time and experience arrive at judgments concerning what is acceptable and what is not acceptable in terms of risks.

An expression of the historic approach is given in Figure 3. This expression is based on historic annual probabilities of failure (high consequence events) and the consequences associated with the failures. The probabilities of failure are actuarial in the sense that they are based on the statistics associated with the failures of systems in the past. They are not notional in the sense that they are analytically derived or computed. The consequences are expressed in terms of U.S. dollars (1984) and deaths. Note that the expression indicates that a statistical death equates to about U.S. \$1 millions.

In Figure 3, note that fixed drilling rigs have a $Pf \approx 1$ E-3 per year. This is due to all causes. Further examination of the causes indicates that about 20% can be attributed to 'natural' causes; the rest are due to 'accidents' (e.g. blowouts, collisions, fires, explosions). Thus, Pf for natural hazards would be about 2E-4 per year.

Another challenge associated with this approach regards the uncertainties and variabilities that are incorporated into this actuarial data based approach; there are no Type II uncertainties. Modeling variabilities are not present in actual tests (the median or central tendency values are!). Thus, when this approach is used, the information must first be carefully evaluated to eliminate Type III uncertainties (those due to human and organizational factors), and then it must be realized that the remainder represents fundamentally Type I uncertainties (due to natural variabilities). It is important there is a consistent treatment of the reliability targets with the analytical processes that are used to help demonstrate that these targets can be achieved. It is for this reason, that some reliability analysists favor not including the additional Type 2 uncertainties – but, they do favor including the necessary corrections to the predictive analytical models to assure that they develop realistic results; this means that the Bias central tendency corrections must be included.

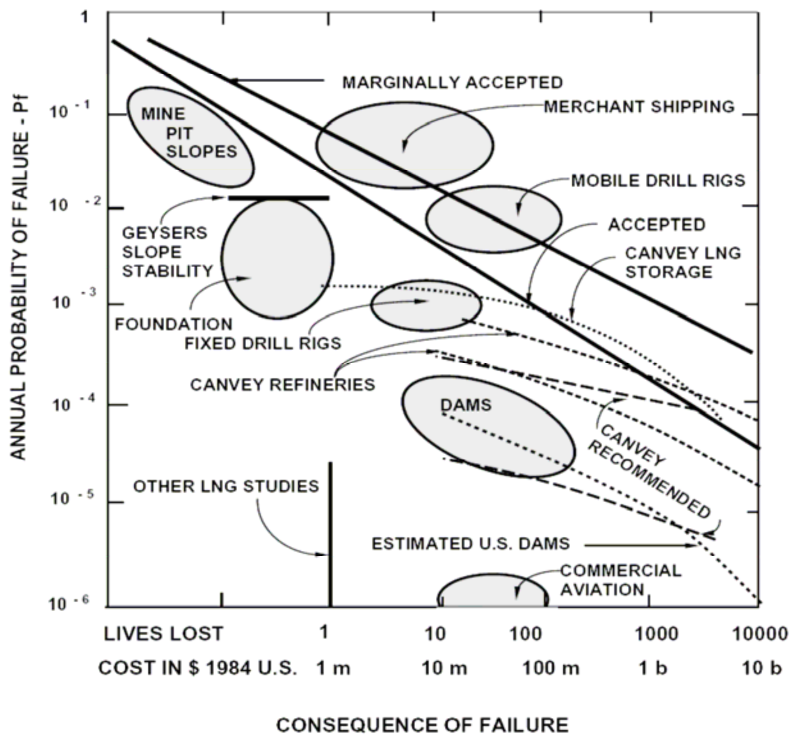

Figure 3- Historic risk tradeoffs

Typical collapse failure rates for engineered public structures like domestic housing, commercial office buildings, and bridges in the U.S., Canada, western Europe have been estimated to be in the range of 2E-5 per year.

Another expression of historic risk tradeoffs is that of the FAR (Fatal Accident Rate). The FAR is the number of fatalities per 10^8 (10E8) hours of exposure to given activities. Figure 4 summarizes some current FARs for activities in the U.K. Commercial industrial activity FARs range from 4 (chemical processing) to as high as 250 (commercial aviation). The commercial aviation FAR increases by factors of up to 10 at different locations around the world (Africa and China have the highest FARs).

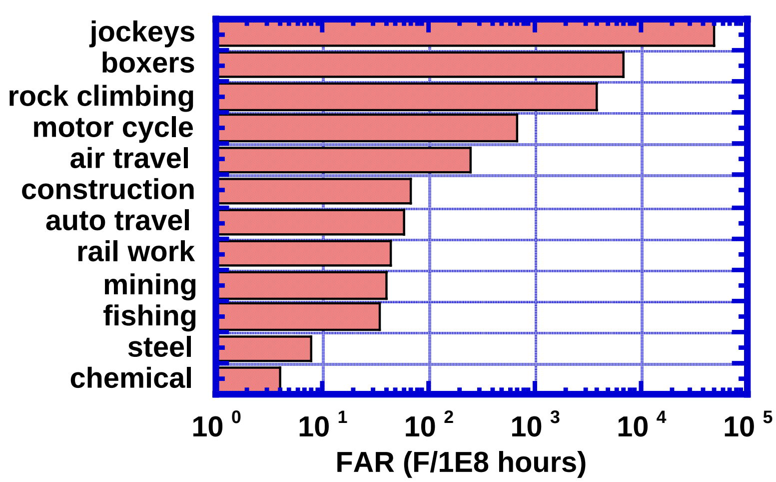

Figure 4 - General activity fatal accident rates

Figure 5 summarizes the FAR for onshore and offshore exploration and production (E&P) workers worldwide during the period 1988 – 1992 (about 75% of the hours were onshore). The E&P FAR range from a low of about 3 to a high of about 40. Operations in the North Sea area and U.S. have the lowest FAR (average of 5 to 6). Operations in South America and Africa have the highest FAR (average of 20 to 25).

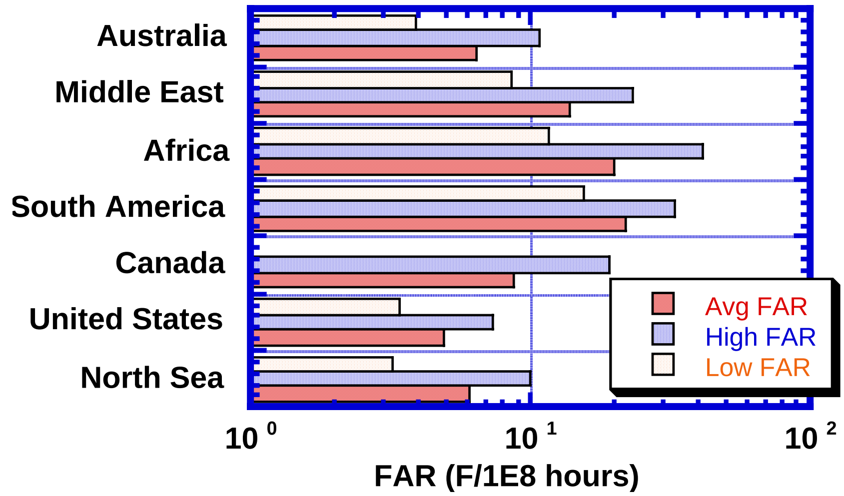

Figure 5 - Worldwide exploration and production fatal accident rates 1988 – 1992

Figure 6 summarizes FAR data on onshore and offshore North Sea based construction as developed by the E&P Forum (1996). Construction activities for fabrication, assembly, and installation are shown for the period 1987 – 1991. Offshore installation is indicated to be the highest with an average FAR of 12 followed by offshore assembly FAR of 10. All onshore construction related average FAR are less than the offshore construction related average FAR.

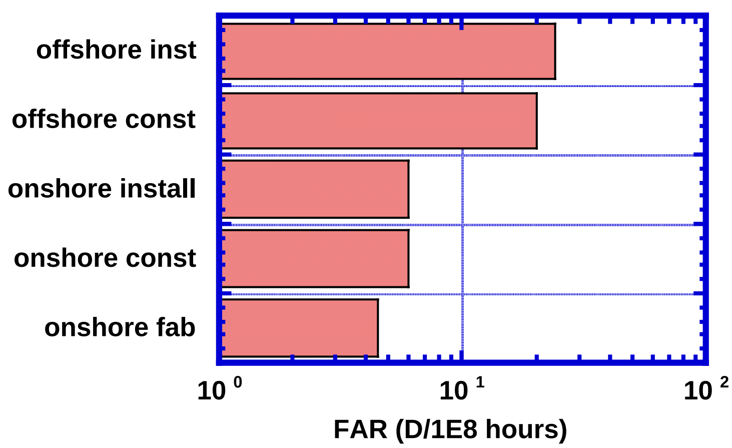

Figure 6 – North Sea construction related mean FAR 1987 - 1991

Figure 7 summarizes mean FARs for 1991-1998 for UK sector offshore activities. The highest mean FAR is that associated with air diving operations (21) followed by maintenance operations (5.5). The air diving FAR is not considered to be reliable because there was no estimate provided for the hours of diving exposure, only for the number of divers employed in offshore operations.

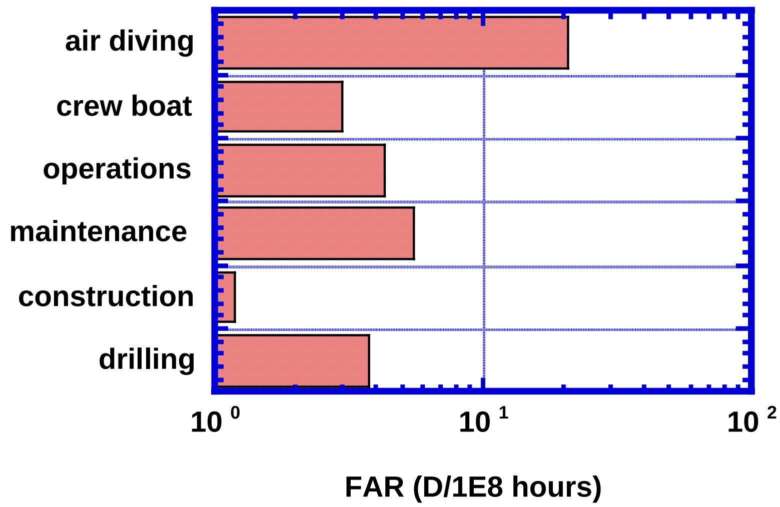

Figure 7 – UK offshore operations mean FAR 1991 - 1996

The U.K. oil and gas operations Employer Incident Analysis report provided the best data that could be located during this study regarding diving accidents. This report provided information on diving fatalities and very serious injuries (single category) and serious injuries (over 3 days lost work). During this 8 year period, there was a total of 16 fatalities and very serious injuries and 43 serious injuries associated with diving accidents.

The number of divers working in the UK offshore oil and gas operations during the period 1991 – 1998 ranged from 900 to 1300. However, no data was provided on the time divers spent offshore nor on the time they spent in underwater operations. The DNV Technica report covering UK offshore operations during this time period indicated that there were on the average about 17,000 to 18,000 air dives per year. If the figure of 18,000 air dives per year and 3 hours of exposure during each of these dives, the FAR plus very serious incident rate would be 3,700E-8 per hour and the SIR would be 100E-6 per hour.

The DNV Technica analysis included an estimate of diving fatalities associated with UK oil and gas operations of about 0.3 per year. Using the same figures for dives per year and hours per dive would result in an FAR = 556 E-8 per hour of air diving exposure. This figure agrees well with that provided by the E&P forum of FAR = 580.

Standards of Practice Approach

A third approach is that of the ‘standards-of-practice.’ Standards-of-practice are represented in the design codes and guidelines that are utilized by organizations and industries. Standards-of-practice are also included in the decisions that are made by system owners / operators and the associated regulators to design, requalify, and operate such systems.

Codes and guidelines can be analyzed using reliability based models to determine the reliabilities inherent in the codes and guidelines. The American Society of Civil Engineers (ASCE) Standard “Minimum Design Loads for Buildings and Other Structures” defines four categories of buildings and other structures that represent increasing hazards to human life in the event of failure (Table 1).

Table 1 – Classification of buildings and other structures for definition of design loadings (pa = per annum)

Occupancy	Category
Low hazard to human life (agriculture facilities, temporary and storage facilities)	I (10^{-2} pa)
Structures except Categories I, III, and IV	II (10^{-3} pa)
Substantial hazard to human life (more than 300 occupancy buildings, more than 250 occupancy schools, more than 500 occupancy adult education facilities, more than 50 occupancy health care facilities)	III (10^{-4} pa)
Essential structures (hospitals, emergency facilities, power facilities)	IV (10^{-5} pa)

For example, for wind loadings, Category II facilities have an importance factor, I, of 1.0. The design wind loading are associated with an annual probability of 0.02 or an average return period of 50 years (98 %tile, 2.05 standard deviations from the median). Given an uncertainty in the annual maximum wind loadings of: σ_{lnV}. = 0.72 ln ($V_{10,000\ yr}$ / $V_{100\ yr}$) = 0.72 ln (250 mph / 125 mph) = 0.50. The wind force varies as a function of the square of the wind speed, thus the uncertainty in the wind force could be estimated as: σ_{lnD}. = 2 (0.5) = 1.0. The ratio of the design wind force to the median annual maximum wind force would be: B_{D50} = exp (2.05 x 1.0) = 7.8. Given a design factor of safety of FS = 2.0, a capacity median bias of B_{C50} = 1.5, a design force median bias of B_{D50} = 0.67, and an uncertainty in the structure capacity of σ_{lnC}. = 0.25, the annual Safety Index could be determined from: β = ln (7.8 x 2.0 x 1.5 x 1.5) / 1.03 = 3.5, or an annual probability of failure of Pf ≈ 2.3 E-4. The average return period of the wind force that would bring the structure to its ultimate limit state would be approximately 5,000 years. Category III and IV structures would be designed to have greater capacities and reliabilities.

A standard-of-practice approach to avoid some of the problems associated with the economics and historic data based approaches. An expression of the standard-of-practice approach is given in Figure 6. The probabilities of failure are computed or notional and include only Type I or 'natural' (inherent randomness). The probabilities of failure are the total Pf and include both intrinsic (environmental, natural) and extrinsic (human, organizational) caused failures. The consequences of failure include the best estimates of economic costs associated with loss of property, injuries, restoration, productivity, and resources. The data points shown are for new and existing platforms that have been evaluated to determine their risks. The two lines labeled acceptable and marginal are those placed by the author based on the decision making processes that have accepted or rejected the assessed risks.

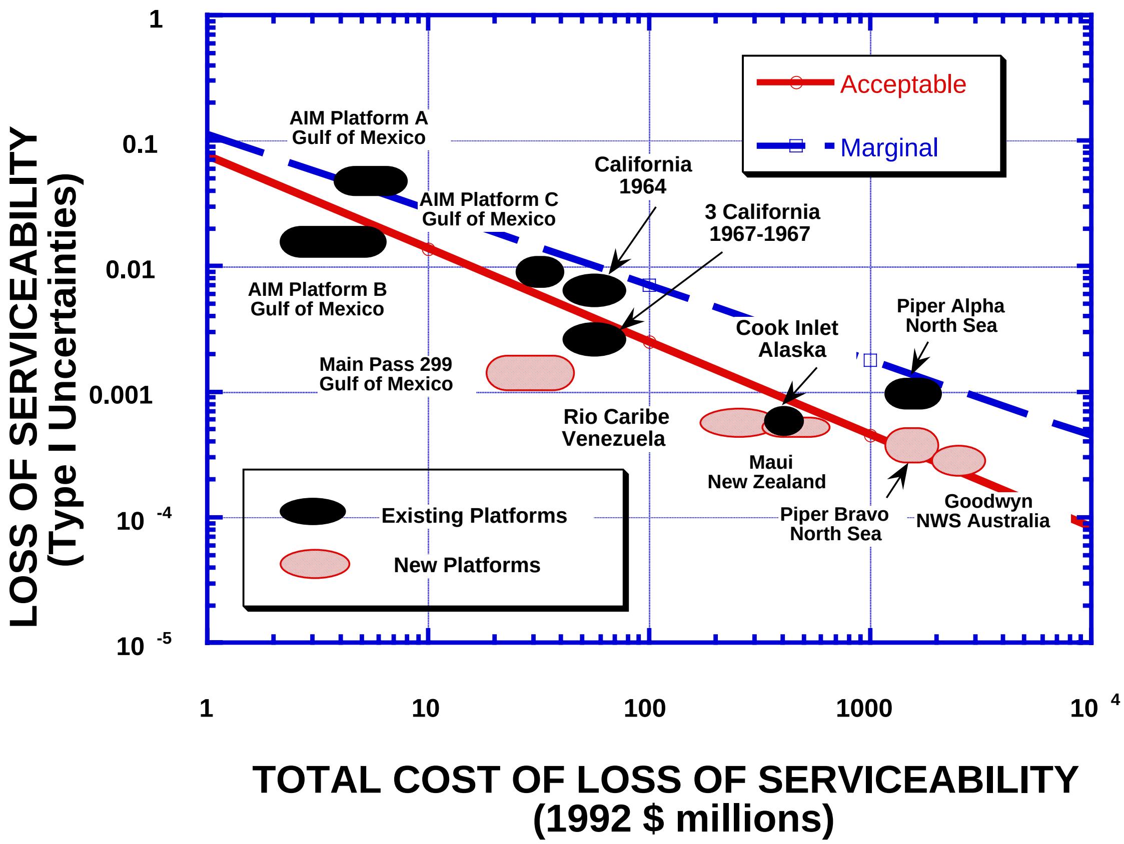

Figure 6 - Standard-of-Practice Approach

The U.K. Health and Safety Executive (HSE) have stipulated an individual risk of 1E-3 per year of exposure to the activity as a maximum tolerable criterion for workers. This individual risk would translate to an FAR = 500 (exposed 50 weeks per year, 40 hours per week, 2000 hours per year). An individual risk of 1E-4 per year is used by the HSE as a maximum tolerable criterion for members of the public from any large-scale industrial hazard (FAR = 50). These criteria have been proposed for application to average individual risk on offshore installations as 1E-3 per year for the maximum tolerable for installations in general and 1E-4 per year for new installations.

Shell has published guidelines for assessing individual worker risks (1993): if above 1E-3 per year (exposed to the hazardous activity), fundamental improvements are needed and only to be considered if there are no alternatives and people are well informed; if 1E-4 to 1E-3 per year significant effort required to improve; if 1E-5 to 1E-4 per year investigate alternatives; if 1E-6 to 1E-5 per year consider cost-effective alternatives. British Petroleum (1995) uses an ALARP approach with a maximum of 1E-3 per year for the most exposed workers.

In development of design guidelines for systems to be located offshore Canada, the Canadian Standards Association (CSA) based the guidelines on two Safety Classes: Safety Class 1 involved great risk to life or high potential for environmental pollution or damage in the event of failure; Safety Class 2 involved small risk to life and low potential for environmental pollution or damage. Annual probabilities of failure of 1E-5 and 1E-3 were stipulated for these two Safety Classes. The CSA specified the average return periods for three classes of loadings that could act on the offshore structures: 1) frequent environmental processes 100 years (with a load factor of 1.35), 2) rare environmental events 1,000 to 10,000 years (with a load factor of 1.0), 3) accidental loading events 1,000 to 10,000 years (with a load factor of 1.0).

In the study of alternatives for decommissioning three offshore platforms in the Pacific OCS Region (Twachtman Snyder & Byrd 2003), a most probable FAR = 600 was used for air diving operations with low bound FAR = 500 and high bound FAR = 700. The diving operations associated with removal of the deep water platforms were estimated to range between approximately 1,200 hours and 2,300 hours (depending on method used to section and remove the platforms).

Excavation Slope Factors of Safety

Target Reliability

Information has been requested from Taylor Energy Company to develop estimates of the ‘exposures’ that are associated with the proposed excavation and well plugging and abandonment project. This information has not been received at this time (December 30, 2007). In lieu of this information, estimates will be used to illustrate how the target reliability for this project might be developed.

For the economics model that has been presented in this report, the costs associated with failure (slope instability resulting in filling the excavation), have been guesstimated to be CF = \$110 million. The cost required to reduce the probability of failure by a factor of 10 has been guesstimated to be ΔCi = \$4.5 million. The resulting Cost Ratio would be Rc = 24.4. The annual present value function has been guesstimated to be pvf = 5 years. The resulting guesstimated effective life of the system would be 122 years.

The resulting ALARP probability of failure would be Pf = 3.5 E-3 per year (reliability of Ps = 99.6 % per year). Given a normal working exposure of 2,000 hours per year, this would equate to an FAR = 1750.

The Historic risk tradeoffs (Figure 3) indicate an ‘acceptable’ probability of failure (single death) of about 1 E-2 per year or a FAR = 5000 (2000 hours per year conventional exposure). Data from exploration and production operations in the U.S. and North Sea indicate maximum FAR in the range of FAR = 20 to 30.

The Standards of Practice characterized with HSE and Shell guidelines indicate ALARP probabilities of failure in the range of 1 E-3 per year of exposure to the hazardous activities. Given that the ICAF were \$10 million per life, the Standard of Practice approach expressed in Figure 6 indicates a Pf $\approx$ 1 E-2 per year.

Based on the foregoing results, for the purposes of these preliminary analyses, Target Reliabilities of 99% per year to 99.9 % per year (Pf = 1 E-2 to E-3 per year of exposure to the hazardous activity) will be used to derive the excavation slope Factors of Safety. A normal year of exposure will be based on 2,000 hours per year. This equates to FAR = 500 to 5000.

Given that a diver working on this project will be exposed approximately 500 on-bottom hours in a year, the Target Reliabilities would be 96 % per year to 99.6 per year (Pf = 4 E-2 to 4 E-3 per year). This is equivalent to an annual Safety Index of β = 1.75 to 2.62.

Uncertainties

Based on the information provided by FMMG, the uncertainty in the soil shear strength effective at resisting the long-term gravity induced shear stresses, is estimated to be $\sigma_{lnR} = 0.30$.

Based on the information provided by FMMG, the uncertainty in the long-term gravity induced shear stresses is estimated to be $\sigma_{lnS} = 0.20$.

Based on the information provided by FMMG, the correlation coefficient relating the magnitudes of the long-term gravity induced shear stresses and the associated effective soil shear strengths is estimated to be $\rho_{RS} = -1$. There is an inverse correlation between the demand shear stresses and the capacity shear strength: as the effective shear stress increases the effective shear strength drops due to creep strain degradation effects.

The total uncertainty in the demand and capacity variables would be $\sigma_{lnRS} = 0.50$.

Biases & Non-Static Demand Effects

No detailed studies have been performed to evaluate the potential Biases (true value / nominal or predicted value) associated with the demand slope induced shear stresses and the capacity shear strengths. No detailed studies have been performed to evaluate any potential effective loading factors that would address time-varying demands and non-linear soil capacities.

Until detailed studies can be performed to determine the Biases, based on the current analytical model being used by FMMG in determining the slope stability, the median Biases in the demand and capacity variables are estimated to be $B_{S50} = B_{R50} = 1.0$.

Until detailed studies can be performed to determine the effective loading factor, based on the current analytical model used by FMMG in determining the excavation slope stability, the median non-static demand or loading effect is estimated to be $Fe_{50} = 1.0$.

Factor of Safety for Excavation Slope Stability

Given the foregoing results, if there were no truncations in the demand and capacity variables, the required Factor of Safety for the excavation slope stability can be calculated as follows:

$$FS = (1.0 \times 1.0 / 1.0) \exp (1.75 \text{ to } 2.62 \times 0.5) = \mathbf{2.4 \text{ to } 3.7}$$

Given the foregoing results, if there were truncations in the capacity variable that are developed when the excavation is made to develop a defined slope which is accurately measured and allowed to ‘age’ to confirm that the slope is stable, the required Factor of Safety for the excavation slope stability can be calculated as follows:

$$FS = (1.0 \times 1.0 / 1.0) \exp (1.75 \text{ to } 2.62 \times 0.2) = \mathbf{1.4 \text{ to } 1.7}.$$

Summary & Conclusions

Given that the excavation slopes will be constructed to a given slope which is accurately measured and allowed to ‘age’ to confirm that the slope is stable, **the foregoing information indicates that a Factor of Safety of 1.5 would be acceptable for the proposed well-plugging operations.** The inherent target reliability in this Factor of Safety must be approved by the management of Taylor Energy and approved for this operation by the U.S. Minerals Management Service.

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