Geohazard assessment lifecycle for a natural gas pipeline project

D Lekkakis\textsuperscript{1}, M D Boone\textsuperscript{2}, E Strassburger\textsuperscript{3}, Z Li\textsuperscript{1} and W P Duffy\textsuperscript{1}

\textsuperscript{1} Bechtel Ltd, 2 Lakeside Drive, Park Royal, London NW10 7FQ, UK
\textsuperscript{2} Bechtel Corporation, 50 Beale St, San Francisco, CA 94105, USA
\textsuperscript{3} Bechtel Corporation, 3000 Post Oak Boulevard, Houston, TX 77056, USA

Email: Lekkakis@Bechtel.com

Abstract. This paper is a walkthrough of the geohazard risk assessment performed for the Front End Engineering Design (FEED) of a planned large-diameter natural gas pipeline, extending from Eastern Europe to Western Asia for a total length of approximately 1,850 km. The geohazards discussed herein include liquefaction-induced pipe buoyancy, cyclic softening, lateral spreading, slope instability, groundwater rise-induced pipe buoyancy, and karst. The geohazard risk assessment lifecycle was comprised of 4 stages: initially a desktop study was carried out to describe the geologic setting along the alignment and to conduct a preliminary assessment of the geohazards. The development of a comprehensive Digital Terrain Model topography and aerial photography data were fundamental in this process. Subsequently, field geohazard mapping was conducted with the deployment of 8 teams of geoprofessionals, to investigate the proposed major reroutes and delve into areas of poor or questionable data. During the third stage, a geotechnical subsurface site investigation was then executed based on the results of the above study and mapping efforts in order to obtain sufficient data tailored for risk quantification. Lastly, all gathered and processed information was overlain into a Geographical Information database towards a final determination of the critical reaches of the pipeline alignment. Input from Subject Matter Experts (SME) in the fields of landslides, karst and fluvial geomorphology was incorporated during the second and fourth stages of the assessment. Their experience in that particular geographical region was key to making appropriate decisions based on engineering judgment. As the design evolved through the above stages, the pipeline corridor was narrowed from a 2-km wide corridor, to a 500-m corridor and finally to a fixed alignment. Where the geohazard risk was high, rerouting of the pipeline was generally selected as a mitigation measure. In some cases of high uncertainty in the assessment, further exploration was proposed. In cases where rerouting was constrained, mitigation via structural measures was proposed. This paper further discusses the cost, schedule and resource challenges of planning and executing such a large-scale geotechnical investigation, the interfaces between the various disciplines involved during the assessment, the innovative tools employed for the field mapping, the classifications developed for mapping landslides, karst geology, and trench excavatability, determining liquefaction stretches and the process for the site localization of the Above Ground Installations (AGI). It finally discusses the objectives of the FEED study in terms of providing a route, a ± 20% project cost estimate and a schedule, and the additional engineering work foreseen to take place in the detailed engineering phase of the project.
1. Introduction
The envisioned pipeline project has an approximate length of 1,800 km and a diameter of 48-to-56 inches and comprises of a buried natural gas pipeline system including all above ground facilities. The above ground facilities include temporary facilities such as laydown areas and construction camps as well as permanent facilities including compressor stations, metering stations, off-take stations, block valve stations, telecom and control systems, and other related equipment and installations. The pipeline also includes an offshore portion, the details of which will not be discussed in the present paper. The onshore pipeline is predominately buried within a 3-m deep trench, with the exception of the various crossings (river, road, utility) where environmental, soil erosion or construction constraints may potentially dictate a greater burial depth.

2. Pipeline alignment evolution
The initial route identified was a 2-km wide corridor, routed to satisfy basic geographical constraints such as the gas metering stations at the bordering countries, in-country fixities (off-take stations), known archaeological sites and populated areas, and to accommodate the offshore crossing fixities (landfalls) and to provide the shortest alignment as possible. This first revision was largely a product of experience, running parallel to existing pipelines. As more data became available, the route was updated and the corridor reduced to 500m, within which an alignment was fixed. Once the alignment was fixed, all subsequent changes were recorded in “route change files”, recording the reason for the change, the economic benefits (shortening of alignment, reducing inflection points and thus bends, etc.), whether the change was within the identified 500m or 2km corridors and whether it potentially clashed with constraints such as constructability, geohazards, socioeconomic (settlements and public safety), archaeology and cultural heritage, environmental and authority/stakeholders. The project underwent extensive route development since inception. The finalized “FEED” route was the 8th revision.

3. Geohazard risk assessment lifecycle
3.1. Desktop study
A desk study was performed to describe the geologic setting along the alignment and to conduct a preliminary assessment of the geohazards. The main inputs to the desktop study included aerial photography and Digital Terrain Modelling topography data over a 2-km wide pipeline corridor, geologic maps from government agencies and local authorities, as well as data from open sources (i.e. province boundaries, district boundaries, roads, rivers, etc.). The study output was a preliminary route selection report and a surficial geology report, on which the initial route revisions were based.

3.2. Field geohazard mapping
The field geohazard mapping was conducted with the deployment of 8 teams of geoprofessionals, who spatially covered the 500-m pipeline corridor. State-of-the-art tools such as rugged Tablet PC’s were employed for data collection. These added both to the productivity of the teams and to the accuracy of the data collection, as its embedded GPS functionality allowed swift positioning and orientation and its preloaded GIS layers such as topo and geology maps provided a holistic view of the area to the field personnel. The standardization provided by use of these tools further minimized the post-processing effort and allowed for seamless exchange of information between the field teams and the design office (basemap and route revision updates, embedding field photos). At critical areas, the field teams were supported by Subject Matter Experts (SME) primarily in the fields of Karst and Landslides, with further information provided in Sections 4.2 and 4.3. The field mapping efforts were completed within 2.5 months, including reporting.
Planning and execution of the geotechnical exploration program

Once the pipeline had reached a fixed alignment in the 5th revision of the route (500-m corridor), the geotechnical investigation program could be developed. Due to the size and linear nature of the project, consideration had to be given to the cost and schedule of the investigation, the purpose of the investigations and with a view to what data will be used and in which aspects of the pipeline design. The data would be used to:

- Primarily, identify the trench excavatability. To this end, 3-m deep trial pits were assigned at intervals of approximately 1.5 km via use of conventional rubber tired backhoe. For a proper excavatability assessment, the recommendation is that the trial pit excavator be of similar characteristics (tonnage, bucket size, horsepower, output) as the equipment foreseen to be mobilized during construction. If not specified, subcontractors will tend to mobilize lightweight equipment to achieve faster completion of the investigation works thus skewing the excavatability estimates.

- Determine the subsurface characteristics of potential landslides identified during field reconnaissance (geohazard mapping) that could not be rerouted, their impact on the pipeline in case of failure, and the extent of the civil measures that would have to be employed towards mitigation and/or monitoring (depth of failure surface, lateral extent). Boreholes for landslide investigation were typically deeper than the explorations for trench excavatability (up to 50m depth) and the soil column would be continuously cored to enable identification of thin low strength bands or development of shear surfaces.

- Assess the subsoil conditions at crossing locations; primarily at major river crossings, where the depth of the pipeline invert would increase due to soil erosion/scour considerations. This was done via boreholes and trial pits with the aim of identifying conditions that would impact horizontal directional drilling (HDD) design.

- Investigate alignment sections in which potentially high groundwater table in combination with loose sand deposits could lead to liquefaction. This was performed with a combination of boreholes and piezocone penetration tests (CPTu). Due to the CPTu being limited to soft and loose soils, CPTu’s were only advanced when boreholes and Trial Pits in close proximity revealed presence of loose sandy materials and high groundwater table. A first CPTu was advanced adjacent to an existing borehole or trial pit and the results were immediately evaluated using pre-developed liquefaction evaluation charts. If the resulting analysis indicated significant liquefaction, additional CPTu’s were advanced on each side on the alignment, until a negative result (no significant liquefaction) was obtained or until it was evident in the field or from aerial photography that conditions had changed to preclude liquefaction.

- Investigate the potential Above Ground Installation (AGI) sites, primarily on a high level to assist with site selection, and determine groundwater level and approximate foundation parameters. This was evaluated via conventional boreholes (typically 2 boreholes per candidate site). Refer to Section 4.5 below for further information on AGI site selection process.

- Map the soil electrical resistivity along the alignment for purposes of pipeline corrosion protection design.

As expected for such a large-scale investigation spanning over a wide geographical area, there were severe access and weather constraints to overcome. As the investigation commenced in the summer, the most remote parts of the pipeline, i.e. in mountainous terrain which would become inaccessible during winter, were first investigated. Successful completion of the entire alignment, not just accessible areas, was integral to the overall success of the investigation program. The geotechnical spread mobilized as well as the duration and productivities of the investigations are presented in table 1 below for reference:
Table 1. Geotechnical investigation metrics.

| Investigation type          | Avg. Nos rigs mobilized | Nos. of investigations | Duration (months) | Productivity\(^a\) | Cost (percent of overall programme cost) |
|-----------------------------|-------------------------|------------------------|-------------------|---------------------|------------------------------------------|
| Trial Pits                  | 2                       | 1400                   | 8                 | 4 Nos/rig/day       | 23%                                      |
| Boreholes                   | 10                      | 430 (7,000m drilled)   | 8                 | 6 m/rig/day         | 67%                                      |
| Cone Penetration Tests      | 1                       | 230                    | 4                 | 2.5 Nos/rig/day     | 10%                                      |

\(^a\) Includes downtime due to equipment breakdown, transportation, contractual stoppages, access constraints, inclement weather, time-off. Based on 6-day workweek, 1 shift.

3.4. Data compilation into GIS database

The project utilized GIS to manage the project spatial data, and to provide spatial analysis and mapping support. The GIS consisted of a folder-based geodatabase (.gdb) system to store spatial data in a logical collection of folders. Data are held in various formats to enable the data to be used by various software packages, including CAD, Google Earth, Global Mapper and ArcGIS. All data is projected and stored in file geodatabases using the grid-based WGS84 coordinate system. Following is an indicative list of the layers available to the route engineers:

- Aerial photography, digital terrain model (DTM), 1:2000 scale photogrammetric vector topographic features which could produce 2-m elevation contours,
- Parcel boundary and owner information, authority data (third party pipelines, local government boundaries etc.),
- Physical (topographic), biological, socio-economic, and archaeological field survey and ministry data,
- Data from open sources (i.e. all province boundaries, district boundaries, roads, rivers, geologic maps, etc.),
- Geologic mapping (from public sources as well as from independent consultants),
- Desktop geohazard mapping data of the 500m wide route corridor including field photo records,
- Geotechnical exploration data,
- Seismicity contours (regulator’s, project preliminary Probabilistic Seismic Hazard Assessment, independent studies),
- Pipeline corridor and alignment revisions.

All data were subject to rigorous quality checks to ensure consistency and completeness, documented in terms of their source and receipt date, and released for use on a weekly basis.

4. Study deliverables

4.1. Trench excavatability and dewatering

Two major cost and constructability items affect the pipeline, namely trench excavatability and trench dewatering. The following ground conditions data along the pipeline route were considered:

- aerial photography including vegetation and local surface characteristics
Using the above datasets, a set of criteria was developed, with input from construction personnel which allowed subdivision of the pipeline alignment into a finite number of classes, in terms of the expected excavatability and trench dewatering needs. For each available and relevant investigation along the pipeline route, the soil profile down to 3m depth was subdivided into two predominant material types. Excavatability and dewatering classes were then assigned to each material type. Guidelines and criteria used for classifying each material into dewatering and excavatability classes are presented in the following Sections 4.1.1 and 4.1.2. The exploration as a whole was then assigned a class chosen as the most unfavorable class of the predominant materials within the reach. Finally, the pipeline reach corresponding to each investigation point analyzed was derived based on review of the area topography, orthophotos and route geology report, rather than a simple midpoint to midpoint length selection between investigation locations.

4.1.1. Criteria for dewatering classifications. The developed dewatering classification scheme is shown in table 2 below.

| Class | Representative equipment | Representative groundwater and soil conditions |
|-------|--------------------------|-----------------------------------------------|
| 1     | High Dewatering needs (combination perimeter dewatering system and/or cutoff wall) | If the groundwater table is between 0 and 2m below ground surface, combined with presence of coarse grained, permeable soils (GW, GP, GW-GM, GW-GC, GP-GM, GP-GC, SW, SP, SW-SM, SW-SC, SP-SM, SP-SC) or highly weathered rock in the upper 3m of soil stratification. Exceptions: Bedrock that exhibit low permeability in its weathered state is not considered herein and may be considered in subsequent project phases. |
| 2     | Medium Dewatering needs (bottom dewatering e.g. centrifugal trash pump) | If the groundwater table is between 2 and 4m below ground surface, as well as presence of coarse grained, permeable soils (sands, gravel) or highly weathered rock in the top 3m of soil stratification. Otherwise, if the groundwater table is between 0 and 3m with the presence of medium permeability soils (GM, GC, SM, SC) in the same depth range. |
| 3     | No Dewatering required | If the groundwater table is below 4m from ground surface, or if the top 3m of soil stratification is comprised solely of impermeable soils (CL, ML, CH, MH, OL/OH) or intact or fresh rocks. |
4.1.2. *Criteria for excavatability classifications.* The developed excavatability classification scheme is shown in table 3 below.

**Table 3. Trench excavatability classification.**

| Class | Representative equipment | Representative soil conditions |
|-------|--------------------------|-------------------------------|
| 1     | Trench excavation is carried out by means of conventional excavator only (15-20 ton). | All soils, including boulders/blocks of up to 1000mm maximum dimension as well as soft rocks. |
| 2     | Trench excavation is carried out by combination of bulldozer ripper and excavator. Hydraulic jackhammer may also be used for trenching beyond ripping depth. | Heavily fractured strong rocks, or intact weak rocks (i.e. claystone, sandstone, tuffs, conglomerates, anhydrite) of up to 20MPa Uniaxial Compressive Strength, or soils including boulders/blocks of greater than 1000mm maximum dimension. |
| 3     | Trench excavation is carried out by rock trencher. Hydraulic jackhammer may also be used as an aid. | Intact or fractured rocks of up to 130 MPa Uniaxial Compressive Strength (i.e. obsidian, limestone, marl, agglomerate, marble, shale, gypsum). |
| 4     | Trench excavation is carried out by means of drilling, installation of charges, blasting and finally use of excavator for trench clean-up. | Intact rocks of Uniaxial Compressive Strength greater than 130 MPa (i.e. most igneous rocks such as basalt, andesite, granite, gabbro). |

4.1.3. *Recommendations and design.* The evaluation concluded that approximately 2/3 of the pipeline had to be excavated by conventional means, whereas an approximate 20% required the aid of a ripper (e.g. such as the one attached to the Caterpillar D9 bulldozer). Heavier means of excavation such as rock trencher and/or blasting will be required for an estimated 15% of the alignment. Regarding dewatering, it is expected that approximately 90% of the alignment will be situated in dry trench or trench with low dewatering requirements. The remaining length will require some form of dewatering, depending on the proximity to the water bodies, groundwater level, construction methods and type of soils in which the trench is excavated. These recommendations were formalized in a report which specified, by kilometer post (KP), what type of excavation and dewatering, if any, was likely required.

4.2. *Karst characterization*

4.2.1. *Karst identification and assessment.* The characterization of karst features utilized the classification scheme of Waltham [1], in which surface observations are used to assign karst landscapes into five morphological assemblages ranging from k1 – Undeveloped Karst to kV – Extreme Karst. Much of the karst along the Project alignment is a gypsum karst and with the aid of
the project SME, a site-specific classification scheme was developed for the gypsum karst encountered. This scheme is summarized in table 4.

4.2.2. **Recommendations and design.** The key recommendation in Karst areas is to control drainage so as not to change existing drainage patterns. Such changes will lead to new dissolution and impacts to the existing conditions. As with most geohazards, if the hazard could be reasonably avoided through rerouting, the pipeline alignment was rerouted around the hazard. However, of all geohazards encountered along the pipeline alignment, the karstic zones were the most widespread over a large geographic area which made rerouting impractical. Where crossing of karstic areas was required, a detailed analysis was performed and the pipeline was assumed to be unsupported with the length of the unsupported span given by the length of the intersected karstic area. For example, a karst crossing with low hazard of ground movement, designing for an unsupported section of 25 meters may be appropriate while for a karst crossing with high hazard of ground movement, designing for an unsupported section of 50 meters may be inappropriate.

The result is a generic classification scheme for much of the alignment, in terms of kilometers, but a site-specific classification scheme with detailed descriptions for morphology, the hazard to the pipeline, and mitigation measures, for the region where the karst risk is highest. The general classification scheme from literature and the site-specific classification scheme allowed having higher confidence in the determination of risk to the pipeline and provided a transparent process between observation of morphology to hazard and mitigation.

| Class | Morphology | Karst Geohazard | Mitigation Measures |
|-------|------------|-----------------|--------------------|
| kg1   | **Karst margin**, narrow zone along cliff line boundary | Caves and notches undercut steep slopes, increasing slope failure hazard | Cross at right angles to minimize length within hazard zone |
| kg2   | **Polygonal karst**, upland with network of low bedrock ridges between closely-spaced, soil-floored solution dolines, and may contain large, old, collapse dolines | Settlement and suffosional loss of soil on doline floors, possible new subsidence sinkholes a few metres across, very small hazard of collapses 20m across | Avoid crossing doline floors where possible; control the drainage during construction, and do not let the trench fill become a new conduit |
| kg3   | **Plateau karst**, upland and high ground with no surface drainage, with scattered, large, old, collapse dolines | Small but measurable hazard of collapses up to 20 metres across | Control the drainage; possibly thicken the pipe wall to allow spanning a new collapse |
| kg4   | **Immature karst**, generally on mixed rock sequences | Minor, rare small subsidence sinkholes | Not on route |
Mantled karst, upland areas with thick soils over the gypsum, also some basins on plateaus. Small hazard of suffosion sinkholes that start small but may grow to 10m across. Minimal geohazard.

Polje karst, alluviated poljes and lowland valley floors, many with strips of along cliff margins. Minor in alluviated centers, enhanced solution features along margins. Minimize the lengths across steep marginal zones with undercuts and collapse; Control the drainage.

4.3. Landslide identification and risk assessment

The landslide identification methodology generally followed the classification scheme of Cruden and Varnes [2], slightly modified for the use of the project. The following were recorded during the desktop study and field mapping:

- material type (rock, debris, or soil)
- movement mechanism (fall/topple, spread/flow, slide/complex)
- state (active, and inactive (latent or dormant))
- depth (shallow, <1m; intermediate; 1-6m; and deep-seated; >6m)
- size (very small, <0.1ha; small, 0.1-1ha; medium, 1-10ha; or large, >10ha)

As a measure of reference, the FEED effort increased the previously mapped landslide database by 30%, totalling approximately 300 sites. The majority of the features comprised slides (85%) whereas falls and flows only occupied 15% of the sites. The material type was primarily debris (50%) followed by sites in soil (35%) and lastly sites within rock (15%).

Subsequently, the hazard level is defined as the potential for structural failure of the pipeline. The developed criteria leading to the subject scenario include:

- Lateral displacement – horizontal and/or vertical movement of the pipeline may occur. Generally, welded steel pipelines can accommodate some movement. If the soil moves more than 5m, failure was assumed.
- Spanning – pipeline failure was assumed if landsliding resulted in an unsupported pipe section greater than 30m.
- Loading – pipe rupture as a result of landslide loading by burial or impact is evaluated.
- Exposure and impact by landslide debris – the sequence of flooding, scour, and debris flow during or after a storm can cause pipeline failure. The risk due to this sequence of events was evaluated for landslide crossings.

Finally, the risk of landslide reactivation causing structural failure to the pipeline within the typical 50-year project design life is assessed. Reactivation during construction may occur by removal of material at the toe, equipment-induced vibrations, by increasing the driving load at the slide crown, and/or by poor construction surface water retention. Other reactivation scenarios include increasing the pore-pressures by rainfall, and seismic ground shaking. The risk was assessed via a risk matrix scheme (based on the proposals of Lee, Jones and Charman [3], [4], suitably modified for the project), depending on proximity to the pipeline, position of the pipeline relative to the landslide, activity state of the landslide, and the thickness of the landslide deposits crossed by the pipeline (refer to tables 5 and 6).
and 6). The qualitative classification scheme then determines the landslide as one of the following risk categories: none, negligible, low, medium, and high.

Table 5. Landslide hazard assessment matrix for active slides. Risk level corresponds to 50-yr design life.

| Pipeline position relative to slide | 0 to 5m | 5 to 10m | 10 to 25m | 25 to 50m | Greater than 50m |
|------------------------------------|--------|----------|-----------|-----------|-----------------|
| Upslope of backscarp               | High risk | Low risk | Low risk | Negligible | No risk         |
| Main body or beyond its lateral flanks | High risk | Low risk | Negligible | No risk | No risk |
| Downslope of toe                   | High risk | Low risk | Negligible | No risk | No risk |
| Run-out apron                       | Low risk | Negligible | No risk | No risk | No risk |

Table 6. Landslide hazard assessment matrix for non-active slides. Risk level corresponds to 50-yr design life.

| Pipeline position relative to slide | 0 to 5m | 5 to 10m | 10 to 25m | Greater than 25m |
|------------------------------------|--------|----------|-----------|-----------------|
| Upslope of backscarp               | Medium risk | Low risk | Negligible | No risk         |
| Main body or beyond its lateral flanks | Medium risk | Low risk | Negligible | No risk         |
| Downslope of toe                   | Medium risk | Negligible | Negligible | No risk         |
| Run-out apron                       | Negligible | Negligible | No risk | No risk |

The risk identification exercise revealed that 2% of the sites were in the high risk category, 4% in medium risk, 14% in low risk, 26% in negligible risk and 54% in the no-risk category.

4.3.1. Recommendations and design. Nyman et al. [5] recommend that, as a basic mitigation strategy, the pipelines be rerouted to avoid mapped landslides. State of practice, as delineated by Sweeney [6] further enforces routing along ridges as opposed to sideslopes to minimize exposure to landslides. This recommendation was adopted for the project via rerouting around features of high and medium risk levels. Where rerouting is impractical and therefore crossing a landslide was required, other mitigation options including civil measures (retaining structures or ground stabilization) or locally increasing the pipeline burial depth are to be considered during subsequent project design phases. Individual suspect features may also require additional geotechnical exploration or instrumentation monitoring but no such cases were identified during the particular project design phase. Heavy rainfall events and seismic shaking may also generate new, unmapped features which should be identified and risk-characterized during inspections by geoprofessionals throughout the project operational life.

4.4. Liquefaction

4.4.1. Liquefaction identification and assessment. An initial liquefaction screening was made by evaluating all subsurface explorations (borings, piezocone penetration tests [CPTu], and machine
excavated trial pits) to determine if potentially liquefiable soil was present at or below the alignment and if groundwater was within 1.5 meters of the ground surface. In floodplains, the groundwater table was conservatively assumed at the ground surface. The trial pits were helpful to determine locations where rock was encountered at shallow depths (>2m) or where clearly non-liquefiable soils were present, thus used in a qualitative manner. Where the initial screening of liquefaction indicated a potential for liquefaction (soil and relatively shallow groundwater) liquefaction triggering calculations using borehole and CPTu data were performed. One problem encountered in the liquefaction analyses was the very high design peak horizontal ground acceleration (PGA) used in the simplified method estimation of seismic cyclic shear stress (CSR) through the soil column. See Seed and Idriss [7] for a detailed explanation of the simplified method for estimating CSR. The Client provided a preliminary probabilistic seismic hazard analysis (PSHA) indicated regions of PGA equal to 1.6g. At the ground surface this results in a CSR using the simplified method of 1.04 assuming a magnitude 7.5, CSRM=7.5. A recent database of field observations of liquefaction by Boulanger et al. [9] and Boulanger and Idriss [8] report a maximum CSRM=7.5 of 0.686 making the design CSRM=7.5 approximately 50% higher than the largest field observation of liquefaction. Therefore current liquefaction methods require significant extrapolation from field observations to access liquefaction potential for the design seismic hazard. Cyclic laboratory and centrifuge tests summarized by Giannakou et al. [10] indicate that the extrapolation of current liquefaction triggering analyses appears to properly estimate the onset of liquefaction, defined as an excess pore pressure ratio, \( r_u \), defined as the ratio of excess pore water pressure to total vertical stress, equal to 1. Figure 1 presents the cyclic resistance ratio (CRR), or the soil’s resistance to liquefaction, versus clean sand normalized blow count for two liquefaction triggering methodologies along with the case history database from Boulanger et al. [9] and Boulanger and Idriss [8]. The methodology of NCEER [16], Youd et al. [17], and Youd et al. [18] is referred to herein as NCEER. Note that the NCEER method considers any soils with a normalized clean sand blowcount of equal to or greater than 30 too dense to liquefy.

![Figure 1. Comparison of liquefaction triggering methodologies and reported case histories.](image)

### 4.4.2. Recommendations and design.

The hazards from liquefaction are generally pipe buoyancy and lateral spread, leading to lateral deformation of the pipeline. In areas of high groundwater, the pipe is generally designed to be neutrally buoyant meaning that the unit weight of the pipeline in cross-section is near, or slightly above, the hydrostatic pressure. As an added safeguard, tie-downs were
recommended in liquefaction areas. Tie-downs mechanically connect the pipeline to more competent material below the pipeline to resist uplift. In areas of lateral spread, the pipeline should generally be buried beneath the zone of lateral spreading.

For dense sands where liquefaction analyses indicate liquefaction, due to high seismic design inputs, the consequences of liquefaction must be considered. While shaking may cause liquefaction in terms of excess pore pressure ratio, Robertson [11] and Cox [12] show that, in terms of strength, soil state dictates soil strength with loose sands (looser than the Critical State Line, CSL) exhibiting strain softening, moderate density sands (slightly loose of the CSL) tending to exhibit limited strain softening, and dense sands (dense of the CSL) tending to exhibit strain hardening. This process is shown graphically on figure 2. Soils loose of the CSL have a liquefied strength, or shear strength of the liquefied soil, less than that of the static drained strength but as density increases and the soil state crosses the CSL the static drained strength governs. This occurs at approximately a normalized blowcount, \((N_1)_{60}\), for clean sands of 19 to 35 depending on drainage characteristics. Therefore, while liquefaction calculations indicate liquefaction may occur, where the soil is dense enough, or dense of CSL, no mitigation measures are necessary.

Waterman et al. [15] present preferred crossing orientations where lateral spreading is expected for the design seismic input. Generally, the pipeline alignment is adjusted to cross the area of lateral spread perpendicularly to reduce the crossing length and make the lateral movement along the axis of the pipe, or to increase the depth of the pipeline such that the alignment is below the expected depth of the zone subject to spreading.

4.5. Above ground installations site localization

The term AGI refers broadly to all above ground installations and facilities that support the pipeline. The present section refers solely to the Compressor Stations and Metering Stations. The site localization study is essentially the process for determining the exact location of each AGI which is subject to engineering, authority and socio-environmental constraints. Initially, the pipeline hydraulic flow study is used to determine the required general distances along the pipeline at which compressor stations are needed. This study utilizes the pipeline elevation and length data, pipeline diameter(s) and the required flow and outputs the number of compressor stations required to assure the flow, as well as their approximate Kilometric Stations (typically in 50-km ranges). The engineering team would then superpose aerial photography and topography data to identify 5 to 7 candidate sites within the above
range for hosting the AGI in question. The land plots required by the various AGIs were 700m square for the compressor stations and 300m square for the metering stations. These sites would then be ranked according to a series of criteria such as: topography, water and electricity, access, environmental/land use, proximity to settlements, hydrology, and mapped geohazards. The top 3 ranked sites are selected and assigned geotechnical investigations (typically 2-3 boreholes each). A site visit of a multi-disciplinary team follows, comprising of construction, civil, and geo-professionals to confirm the findings of the desktop study and identify any additional hazards/concerns. The sites are then re-ranked according to the findings of the site visit and the borehole findings. Finally, permitting and land acquisition processed take place on the selected site.

5. Scope foreseen to be performed in subsequent engineering phases

While the FEED study is fundamental in providing a project cost estimate (thus aiding the Final Investment Decision) and allowing for an early placement of orders for the long-lead items, the subsequent phase of Detailed Engineering will ultimately finalize the route, provide closure to any outstanding regulatory requirements and be used for the remaining material requisitions. On the geotechnical front, the following studies and exploration programs are foreseen to take place in the Detailed Engineering design:

- Finalizing identification of and performing additional exploration of areas deemed to require further detailed exploration. These included in particular:
  - Reaches of significant realignment of the pipeline, new pipeline segments, and reaches in which exploration could not be performed due to external constraints (ground conditions, military zones, etc.);
  - River crossings where horizontal directional drilling is proposed and the existing data is inadequate for such constructions;
  - Landslide areas which could not be readily routed around, and in which boring under the slide was a consideration;
  - Areas in which the extent of liquefiable material along the pipeline alignment could not be determined from existing explorations and surface features;
  - Fault crossings, which require very specific fault geometry and other design information, and;
  - Above Ground Installations, particularly the compressor stations, which required design level geotechnical investigations.
- Further collection of groundwater levels to provide data over at least a one year duration.
- An update to the initial probabilistic seismic hazard analysis (PSHA) will be required for detailed engineering. The update to the PSHA will use the latest seismic-hazard source model and ground motion prediction equations (GMPE) to calculate the seismic requirements for design along the pipeline alignment for both the pipeline itself and AGIs.
- Fault trenching of 13 active faults along the alignment will be required to identify the precise location, activity, orientation of movement, and distribution of shear for each fault. These data will be used in designing the pipeline fault crossings.

6. Lessons learnt, summary and conclusions

- In order to timely deliver a successful FEED, the engineering party must be very clear regarding the FEED goals, as those are delineated in the contract documents. Usually these goals are defined in terms of a design effort target (approximately 30% design), as well as a cost estimate target (+/-20% cost estimate). Therefore, on the geohazard-driven routing front, attention should be drawn on those elements that may yield major reroutes between FEED and Detailed Engineering such as large landslides, karst landscapes, and less on elements of minor impact on the overall cost, i.e. crossing locations and design depths or adjusting the geotechnical investigation program whenever the route gets revised.
Execution of the geotechnical investigation as early as possible will prove beneficial to the project as it provides valuable excavatability and geohazard information, addressing most of the uncertainty regarding route selection. Route changes that may occur after the investigation has been planned can be easily addressed while the crews are still on site.

Mobilization of expatriate teams and experts was not a straightforward task and required planning ahead. Also, availability and tight schedules resulted in many of the subcontracts to be awarded sole-sourced.

Geotechnical subcontractors, based on their domestic competition and frequency of large-scale projects materializing in their areas of operation, do not follow the same standards in all parts of the world. This became particularly evident in the initial stages of the project, where efforts were concentrated in technical supervision. It is important that unresolved and continuing quality concerns are documented and addressed in the design stage, by increasing factors of safety and looking at the data from a critical viewpoint.

In this project, the Owner hired a separate geotechnical firm to provide field investigation and field quality control, but the contracts were set up to be mutually exclusive, each with the Owner independently. Thus the engineering contractor had no contractual control over the investigation contractor, even though his job was to control the quality of the work produced. Needless to say, this produced significant conflicts between the parties. Additionally, both companies hired independent consultants as their field staff for much of the field works, leading to erratic consistency.

Towards the aim of economizing, the Owner’s contract also stipulated that the investigation contractor would have one field engineer on each of 2 drill rigs, as long as the rigs were within an hour’s travel of each other. Continuous soil core was also stipulated from each boring, which reduced average production to about 6m per day. Blow counts were recorded by the driller, but due to absence of the field engineer much of the time, the quality of the logs did not include field observations normally recorded during drilling. In some cases, work was found to have progressed for 3 days without an engineer’s log having been prepared. Thus, the quality of the final log was suspect and inconsistencies were found in many logs. Sample preservation was also questionable, particularly in adverse weather.

References
[1] Waltham T 2002 The engineering classification of karst with respect to the role and influence of caves International Journal of Speleology 31 19-35
[2] Cruden D M and Varnes D J 1996 Landslides: Investigation and Mitigation Transportation Research Board Special Report (Washington, USA) vol 247 ed Turner A K and Schuster R L (National Academy Press) pp 36-75
[3] Lee E M and Jones D K C 2013 Landslide Risk Assessment (London, UK: Thomas Telford)
[4] Lee E M and Charman J 2005 Geohazards and risk assessment for pipeline route selection Terrain and Geohazard Challenges Facing Offshore Oil and Gas pipelines ed M. Sweeney, (Thomas Telford) pp 95-116.
[5] Nyman D J, Hall W J and Cluff L S. 2006 Observations and Lessons Learned from Three Decades of Seismic Hazard Mitigation for Major Oil and Gas Pipeline Projects Proc. of the 8th U.S. National Conference on Earthquake Engineering (San Francisco, USA, 18-22 April 2006) Paper No. 1016.
[6] Sweeney M 2005 Terrain and geohazard challenges facing onshore oil and gas pipelines (London, UK: Thomas Telford)
[7] Seed H B and Idriss I M 1971 Simplified procedure for evaluating soil liquefaction potential J. Geotech. Engl. Div. ASCE 97 1249–73
[8] Boulanger R W and Idriss I M 2014 CPT and SPT based liquefaction triggering procedures Report No. UCD/CGM-14/01 Center for Geotechnical Modeling, Department of Civil and Environmental Engineering Report, University of California, Davis (Davis, USA) p 134
[9] Boulanger R W, Wilson D W and Idriss I M 2012 Examination and re-evaluation of SPT-based liquefaction triggering case histories Journal of Geotechnical and Geoenvironmental Engineering, ASCE 138 898-909

[10] Giannakou A, Travasarou T, Ugalde J, Chacko J and Byrne P 2011 Calibration methodology for liquefaction problems considering level and sloping ground conditions 5th International Conference on Earthquake Geotechnical Engineering (Santiago, Chile, 10-13 January 2011) Paper No. CMFGI

[11] Robertson P K 1994 Suggested terminology for liquefaction Proc., 47th Canadian Geotechnical Conference (Halifax, Nova Scotia) pp. 277-286.

[12] Cox B R 2006 Development of a direct test method for dynamically assessing the liquefaction resistance of soils in situ PhD. Thesis (University of Texas, Austin).

[13] Hatanaka M and Uchida A 1996 Empirical Correlation Between Penetration Resistance and Internal Friction Angle of Sandy Soils Soils and Foundations 36-4 1-9

[14] Idriss I M and Boulanger R W 2008 Soil liquefaction during earthquakes Monograph MNO-12, (Oakland, USA: Earthquake Engineering Research Institute) p 261

[15] Waterman M, Hundl J D, Wiggs J E, Boone M D, Kottke A K and Sholley, M G 2014 Geohazard mapping and evaluation for a natural gas pipeline project Poster presentation, GSA 2014.

[16] Youd T L and Idriss I M, National Center for Earthquake Engineering Research (NCEER) 1997 Proc. of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils ed Technical Report NCEER-97-022 p 302

[17] Youd T L, Idriss I M, Andrus R D, Arango I, Castro G, Christian J T, Dobby R, Finn W D L, Harder Jr L F, Hynes M E, Ishihara K, Koestor J P, Liao S S C, Marcuson III W F, Martin G R, Mitchell J K, Moriwasaki Y, Power M S, Robertson P K, Seed R B and Stokoe II K H 2001 Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Geoenvironmental Engineering, ASCE 127(10) 817-33

[18] Youd T L, Idriss I M, Andrus R D, Arango I, Castro G, Christian J T, Dobby R, Finn W D L, Harder Jr L F, Hynes M E, Ishihara K, Koestor J P, Liao S S C, Marcuson III W F, Martin G R, Mitchell J K, Moriwasaki Y, Power M S, Robertson P K, Seed R B and Stokoe II K H 2003 Closure to “Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils;” Journal of Geotechnical and Geoenvironmental Engineering, ASCE 129(3) 284-286