| Trans Adriatic Pipeline | Page 1 of 83 | | | | | |
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# Trans Adriatic Pipeline – TAP

**Document Title:**

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| 0B | Issued for Information | Conclusion and recommendations | | 15/07/2015 |
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| 01 | Final issue | Final issue | | 23/11/2015 |
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URS

**TAP - Trans Adriatic Pipeline**

**SOIL INVESTIGATION ITALY**

Prepared for:
E.ON Technologies GmbH

November, 2015

# Geotechnical & Geophysical report - Pipeline (Italy)

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## ATTACHMENTS



## FIGURE WITHIN TEXT

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# 1 INTRODUCTION

This document is the technical report describing geotechnical and geophysical investigation performed preliminarily to the construction of the Trans Adriatic Pipeline (hereinafter referred to as TAP) along the track of the on-shore pipeline in Italian territory (between the Pipeline Receiving Terminal Area and Microtunnel Area), Municipality of Melendugno (LE), and conducted by URS.

URS was assigned of carrying out the above investigation by E.ON New Build and Technology GmbH (hereinafter referred to as ENT), which was component of TAP AG together with STATOIL and AXPO, after awarding the tender for that project.

The reference technical documentation is listed below:

1. Trans Adriatic Pipeline – Geophysical Investigation Italy, 2013;
2. IAL00-ERM-643-Y-TAE-1006 Rev. 00 ESIA Italy: Section 6 Environmental, Socioeconomic and Cultural Heritage Baseline;
3. 2012_10_TAP_Addendum_URS_Proposal_3116048_rev00;
4. IAL00-ENT-000-Q-TSX-0001_00-Description of Area PRT and Pipeline Corridor;
5. IAL00-ENT-000-Q-TLX-0001_00-at01-Soil Investigation Italy - Bill of Quantities;
6. 2012_09_TAP_TSP_Italy_URS_Proposal_3116048_rev00.
7. IAL00-URS-000-Q-TRG-0001_00 - Geophysical Investigation Italy.

The aforementioned documents were therefore used to define:

- the location of survey points;
- the executive procedures of drilling, geophysical surveys and on-site tests;
- depth to be reached with the investigation;
- sampling procedures and delivery of samples to the laboratories;
- geotechnical laboratory tests to be performed in the laboratory.

# 2 PROJECT DESCRIPTION

TAP will transport gas via Greece, Albania and across the Adriatic Sea to Italy and further to Western Europe. Crossing the Adriatic Sea via a 36” pipeline from central-western Albania, the offshore pipeline will get onshore in south-east Italy, Puglia region, and tie in to the Italian existing Snam Rete Gas network.

The project is aimed at enhancing security as well as diversification of gas supplies for the European markets. TAP has also incorporated provisions to accommodate physical reverse flow. The total pipeline length is 871 km approximately.

The pipeline landfall will be on the coast between San Foca and Torre Specchia Ruggeri in the municipality of Melendugno, province of Lecce. The landfall will be constructed using micro-tunnelling technology to minimize the visual and environmental impact on the coastline.

Figure 2-1 shows the TAP – General Overview. The pipeline system in Italy will consist of:

- an approximately 45 km long offshore pipeline, from the Italian jurisdiction boundary (middle of the Adriatic Sea) to the Italian coast,
- an about 8.2 km long onshore pipeline, from the entry point of the offshore microtunnel (KP 0), to the Pipeline Receiving Terminal (hereinafter referred to as PRT, KP 8.203),

and will have an initial capacity of 10 BCM (expandable to 20 BCM) of natural gas per year (around 1.190.000 standard cubic metres per hour).

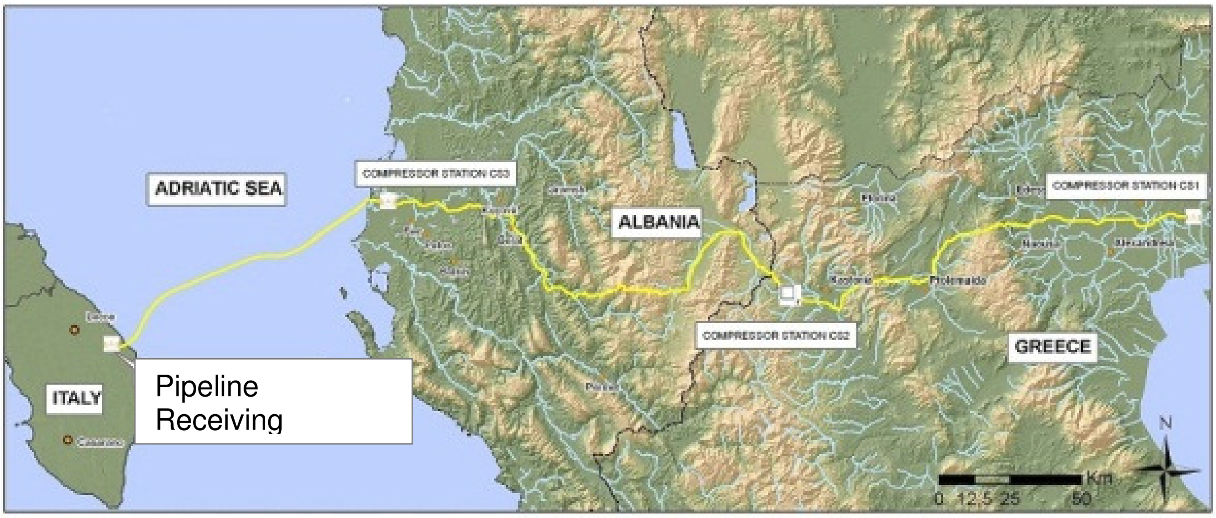



**Figure 2-1: Trans Adriatic Pipeline – General Overview**

The project will also include a Fiber Optic Cable (FOC) to enable communication between PRT, where the supervisory control centre is located, the compressor stations in Albania and Greece as well as the block valve stations installed along the 871 km long pipeline. It shall be laid parallel to the pipeline, along the entire route (onshore and offshore) and will be the primary means of communication between the pipeline stations.

The onshore pipeline (approximately 8.2 Km long from the landfall of the offshore pipeline to the Pipeline Receiving Terminal) runs in an E-W direction in the Province of Lecce to the southeast of the town of Lecce, entirely within the Municipality of Melendugno. The onshore pipeline route will cross a Provincial Road (SP 02 from Lecce to Melendugno), at KP 6.542 and eight more minor municipality road crossings.

The tie-in with the onshore pipeline at the end of this tunnel marks the KP 0 of the onshore route and will be located approximately 600 m off the coast (direction south-west). A Block Valve Station (BVS) is planned to be erected just downstream of this tie-in point.

PRT will be the terminal point of the onshore pipeline and the connection with the Italian national network owned and operated by Snam Rete Gas S.p.A (SRG);

PRT area will also represent the laydown area during construction of the onshore pipeline and the only storage area for pipes for all the construction activities.

The purpose of the terminal inlet section is to receive the incoming gas feed and to act as a point of isolation (and emergency shutdown by means of ESD valves) between the BVS close to the coast and the Terminal itself.

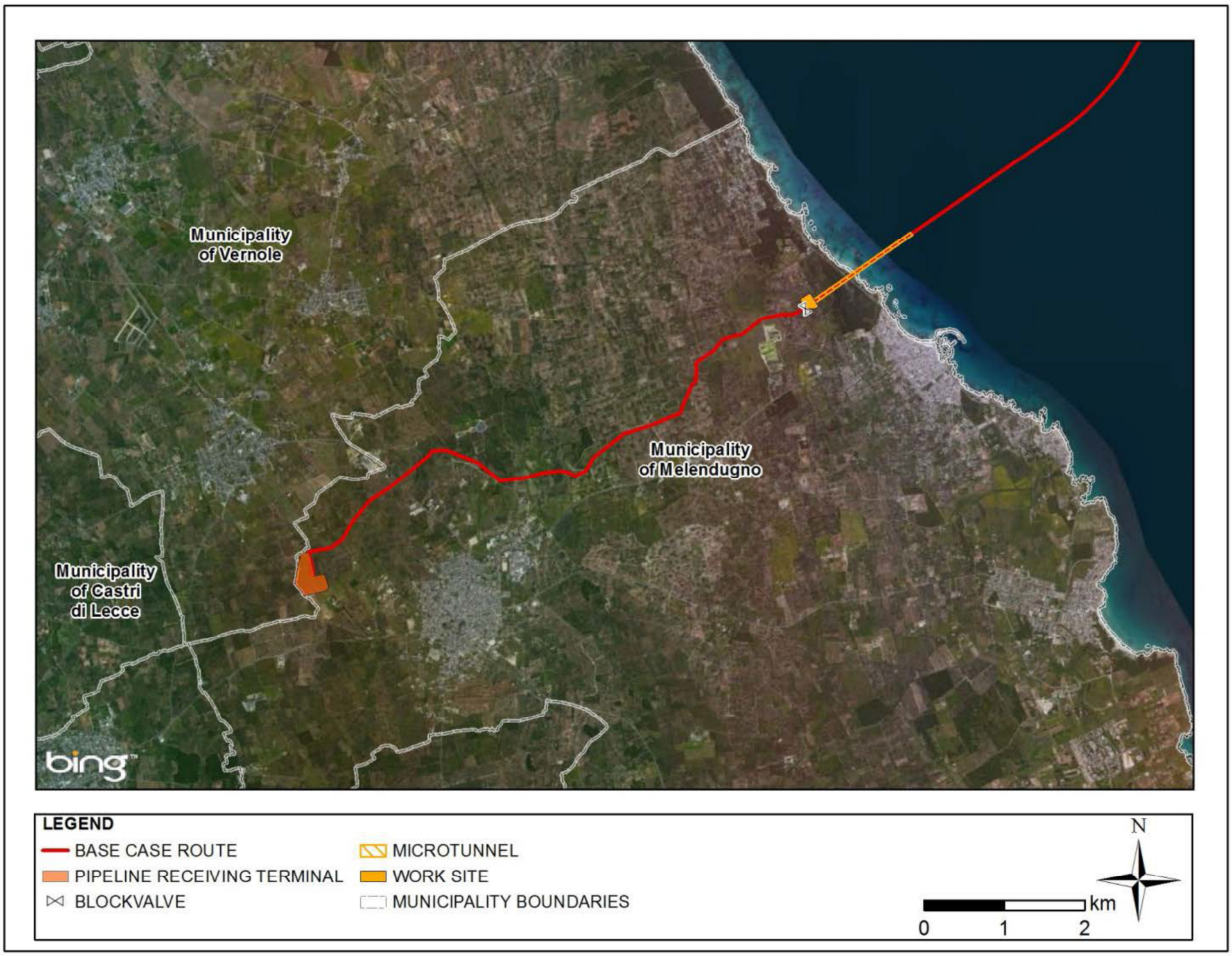



**Figure 2-2: Onshore pipeline route in red (extract from ESIA Italy – Section 4)**

One Block Valve Station (BVS) will be installed close to the pipeline landfall at KP 0.1 in order to enable the isolation of the offshore pipeline from the onshore part for maintenance and safety purposes. The BVS is unmanned and contains as above ground features only a small cabinet for power and control system and a fence to avoid any interference, covering a total surface area of approximately 13 x 14 m (plus surrounding vegetation).

According to Italian regulation (DM 17/04/2008) BVS's in high pressure natural gas pipelines are to be installed every 15 km. BVS's are also required upstream and downstream of railway crossings, at a maximum distance of 2 km between them (DM 23/02/1971). Given the limited length of the onshore pipeline, no additional BVSs are needed.

The BVS will be usually operated remotely from a control centre in the PRT through a fibre-optic cable communication system and will be connected to the local power network. Pipeline block valve, by-pass valves and connected piping will be buried below ground. Valve integrity will be also monitored in continuum by the pipeline Leak Detection System.

The selection of the pipework constituting the BVS is based on the same design standards and specifications used for the selection of the onshore pipeline. The diameter of this pipework will be 12" for the by-pass line and 2" for branches to measuring instruments.

*Source: ENT (July 2013)*

**Figure 2-3: Block Valve Station – 3D Model**

## 2.1 Depth of burial of the pipeline

With regard to the construction methods of gas pipelines in Italy, Ministerial Decree 17/04/2008 prescribes a minimum pipeline cover not less than 0.9 m and 0.4 in rocky ground from the top of the pipe. In any case gas pipelines are usually laid with a minimum cover of 1.5 m in Italy, in order to provide the maximum guarantees of safety from possible interferences with human activities (excavating, ground-breaking for agricultural purposes etc.). Typical trench dimensions respecting the legal requirements can be seen in following figure. TAP AG plans to follow this construction practice and to maintain a minimum cover thickness of 1.5 m.

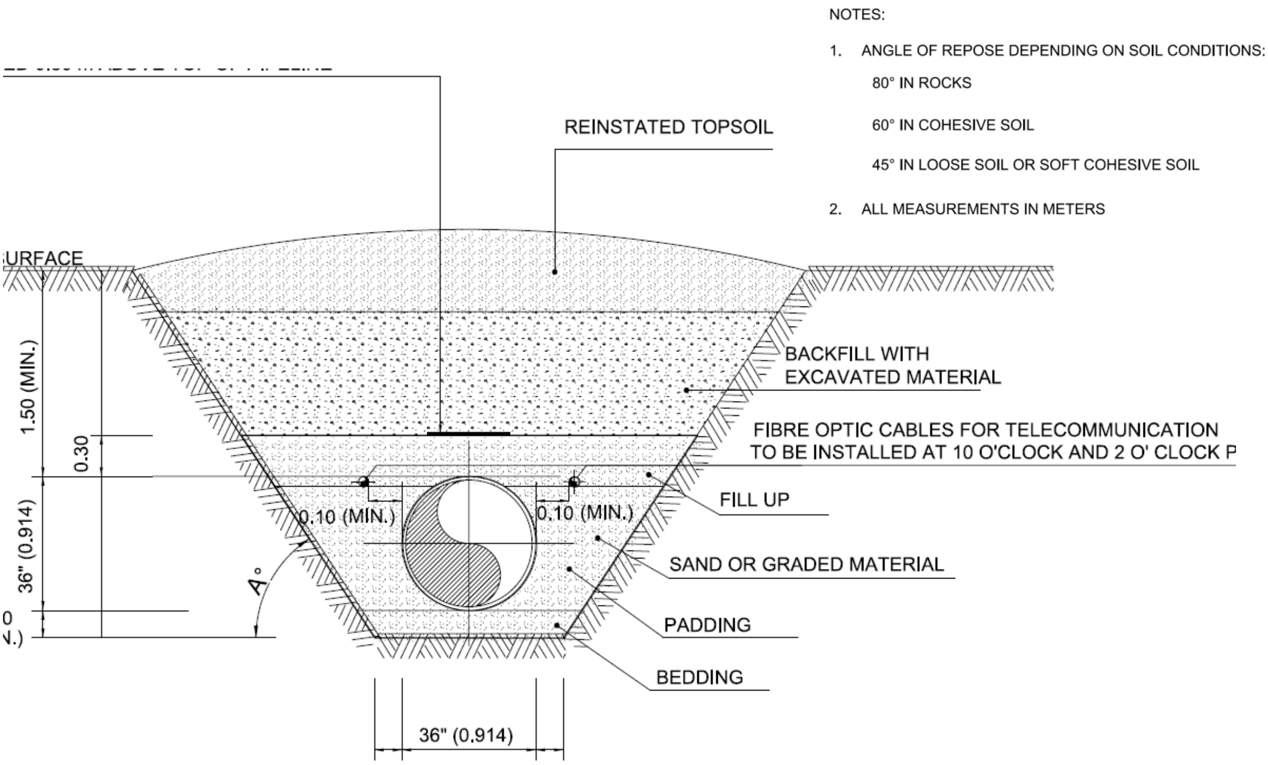



**Figure 2-4: Typical onshore pipeline trench** *(source document: IPL00-SPF-100-F-DFT-0002_01)*

## 2.2 Safety distances and crossings

In accordance with Italian regulations no clusters of houses should be identified within a range of 100 m to the pipeline. In proximity to the planned pipeline route there are only very few single houses, at a distance longer than 20 m (in compliance with the DM 17/4/2008).

In addition to the provincial road and one minor asphalt road crossing upstream of KP 0, there is one more provincial road crossing at KP 6.5 and eight more minor municipality road crossings, summarised in the following Table 2-1 (which includes estimated distance from the landfall tunnel exit Kp). Details of all the asphalt road crossings and the proposed construction method are provided in *IPL00-SPF-100-F-DFT-0002_01* and *IPL00-SPF-100-F-DFT-0009_01*.

**Table 2-1: Crossings of the onshore pipeline**

| ***Crossings Nr*** | ***Crossing Category*** | ***Kp. [km]*** | ***Municipality*** |
|---|---|---|---|
| 1 | SP 366 | - | Melendugno |
| 2 | Secondary road | - | Melendugno |
| 3 | Seasonal road | 0.6 | Melendugno |
| 4 | Secondary road | 1.1 | Melendugno |
| 5 | Secondary road | 2.0. | Melendugno |
| 6 | Secondary road | 4.0 | Melendugno |
| 7 | Secondary road | 4.6 | Melendugno |
| 8 | Secondary road | 5.6 | Melendugno |
| 9 | Secondary road | 5.9 | Melendugno |
| 10 | SP 02 | 6.5 | Melendugno |
| 11 | Secondary road | 7.6 | Melendugno |

The crossings are implemented as small stand alone “worksites” that come into operation as the line progresses. The crossing installation methods are different and can generally be carried out by trenchless (tunneling or boring) or open-cut techniques, with or without casing pipe. The choice of the installation system depends on a number of factors, including: laying depth, presence of water or rock, intensity of traffic, authority requirements, etc.

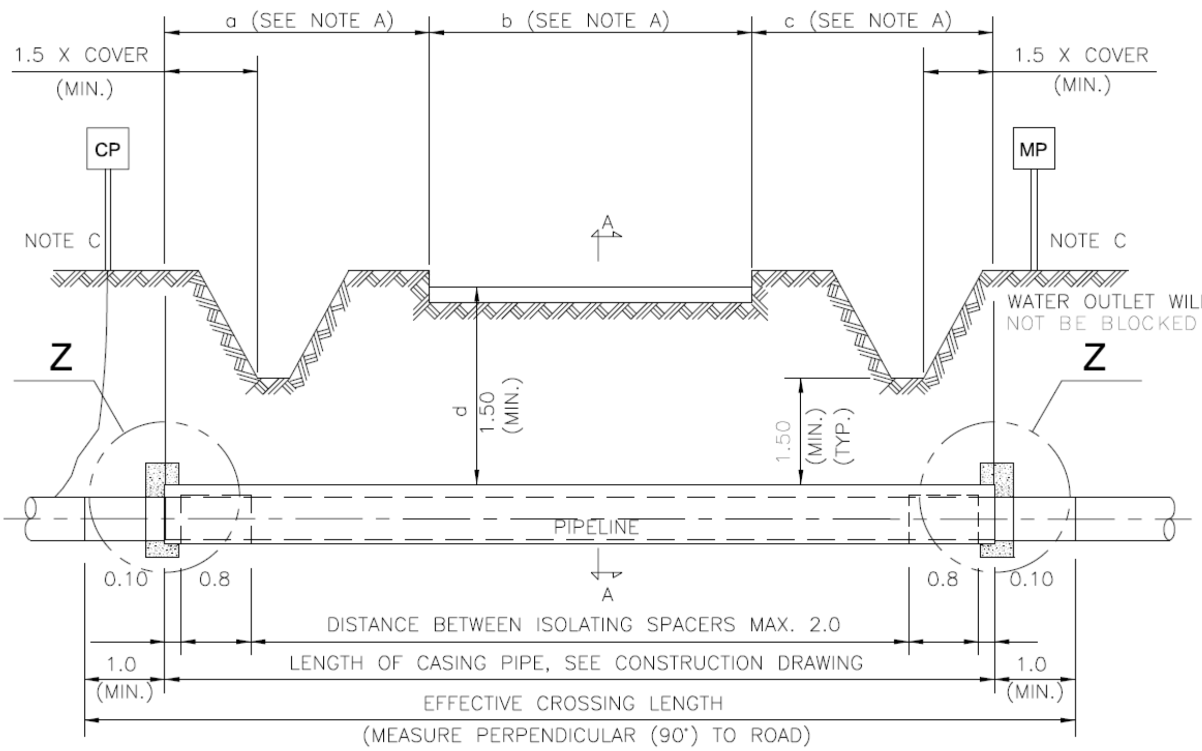



**Figure 2-5**: **Typical crossing of Provincial Road** *(source document: IPL00-SPF-100-F-DFT-0009_01)*

## 2.3 Support of the pipeline in the BVS

Figure 2-6 and Figure 2-7 show the typical support of the pipeline in the BVS: a base plate (PTFE side: up) to be installed by using jackscrews with the pipe to be positioned exactly in the middle of the slide plate. Figure 2-7 shows the slab support for the BVS.

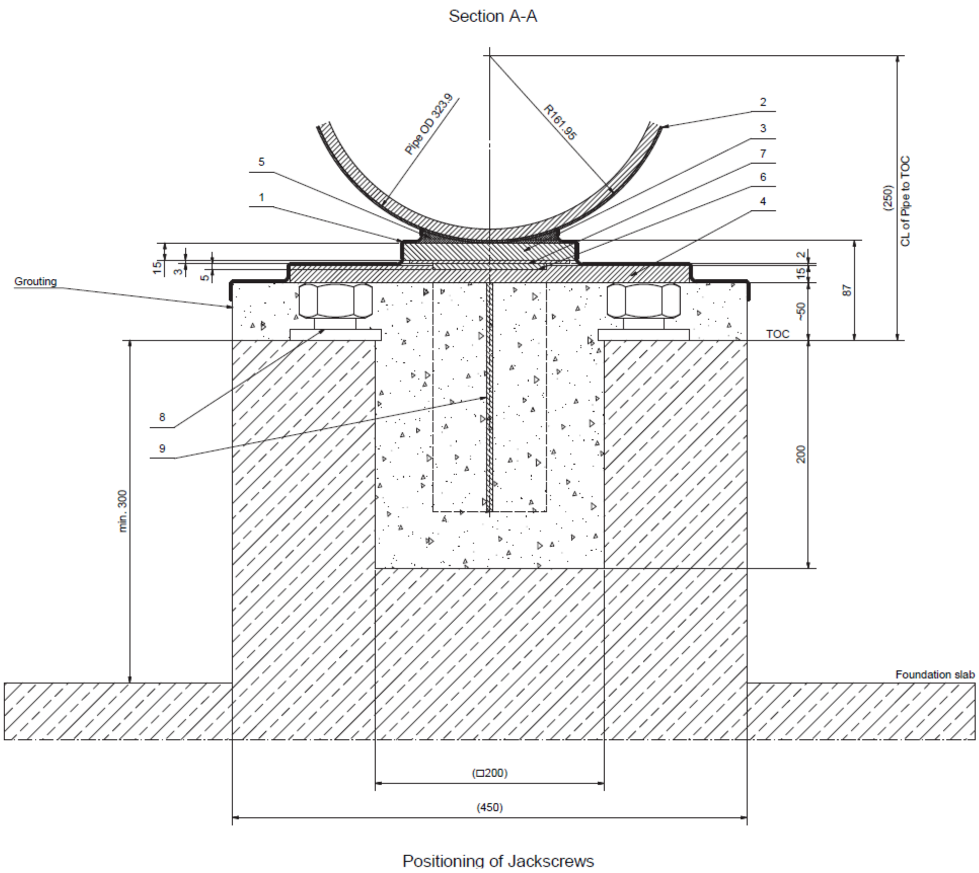



**Figure 2-6: Typical Support, Pipe Saddle, slide support on concrete, below ground, DN 300 (Source CAL00 - ENT - 360 - M - DLT – 0379)**

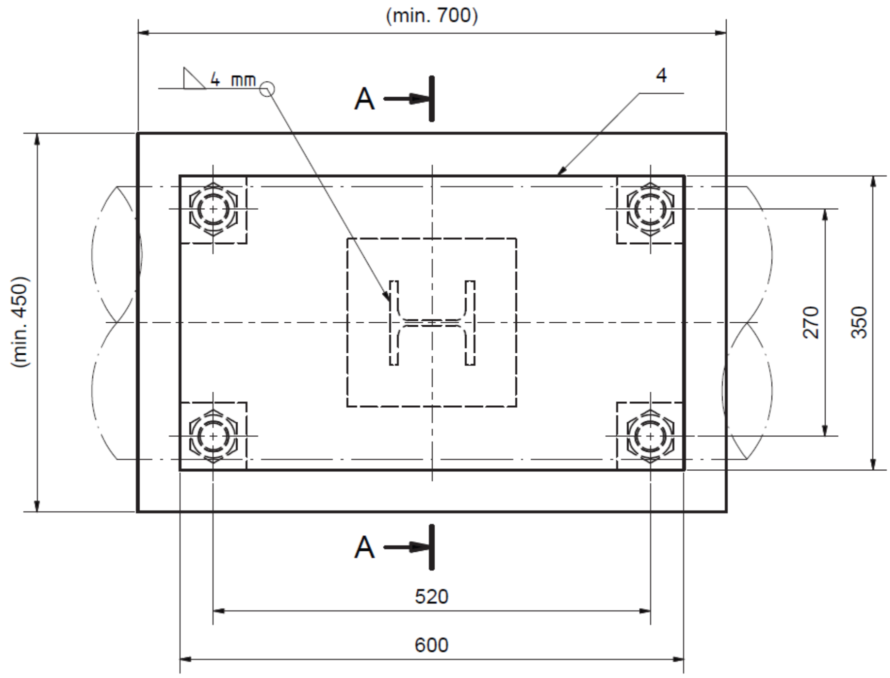



**Figure 2-7: Positioning of Jackscrews (Source CAL00 - ENT - 360 - M - DLT – 0379)**

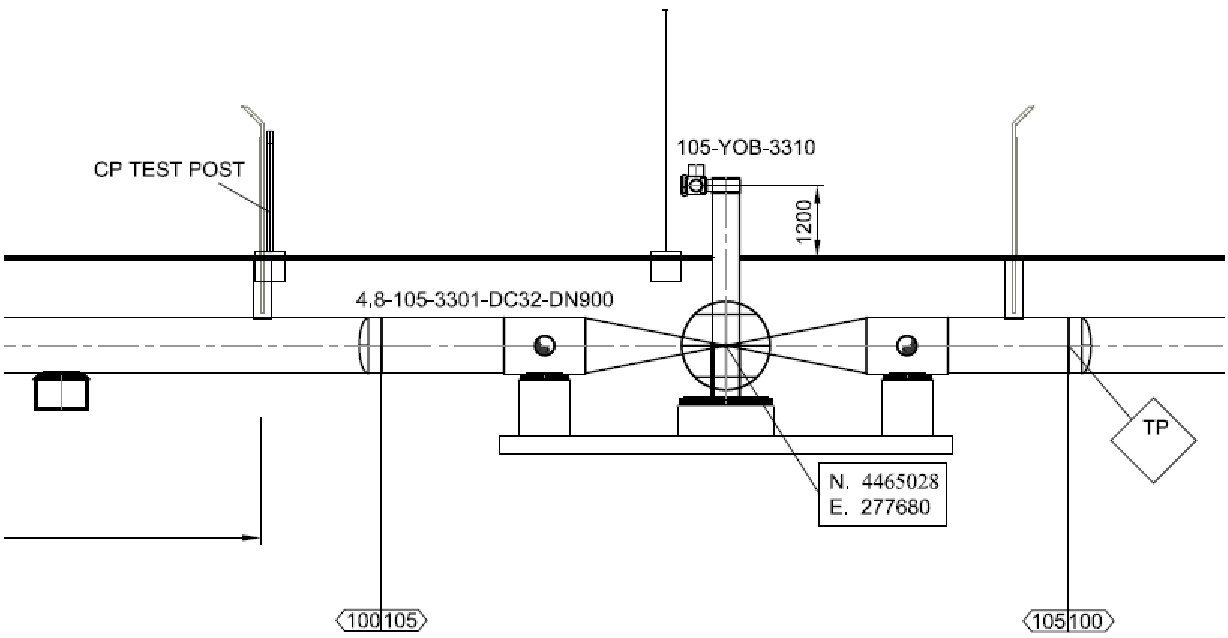



**Figure 2-8: Details of slab foundation and pipe saddle for the BVS (Source CAL00 - ENT - 360 - M - DLT – 0379)**

# 3 SITE DESCRIPTION

The planned pipeline route passes south of a large topographical depression consisting of a wetland named “Palude di Cassano” (Cassano Marsh), which is under environmental protection (Melendugno Municipality Plan).

From the first open-cut crossing with the “Strada Comunale S. Niceta” at KP 0.6 (south-east of the wetland), the pipeline route runs parallel to this paved municipal road for approximately 3.5 km. In order to minimize impact on properties and landscape it changes the side of this road three times more, at KP 1.1, KP 2 and KP 4. The route continues its course mainly through olive groves seeking the side of the road where possible, crossing another provincial road, the ”Strada provinciale Lecce Melendugno” (SP2) at KP 6.5. At a total onshore route length of approx. 8.2 kilometres, the pipeline reaches the PRT area west of the township of Melendugno. This terminal station will be situated close to the border between Melendugno and Vernole, approximately 1.5 kilometres south of the provincial road connecting these towns.

The examined section does not present particular problems and the morphology of the territory predominantly consists of tilting plains, locally slightly undulating.

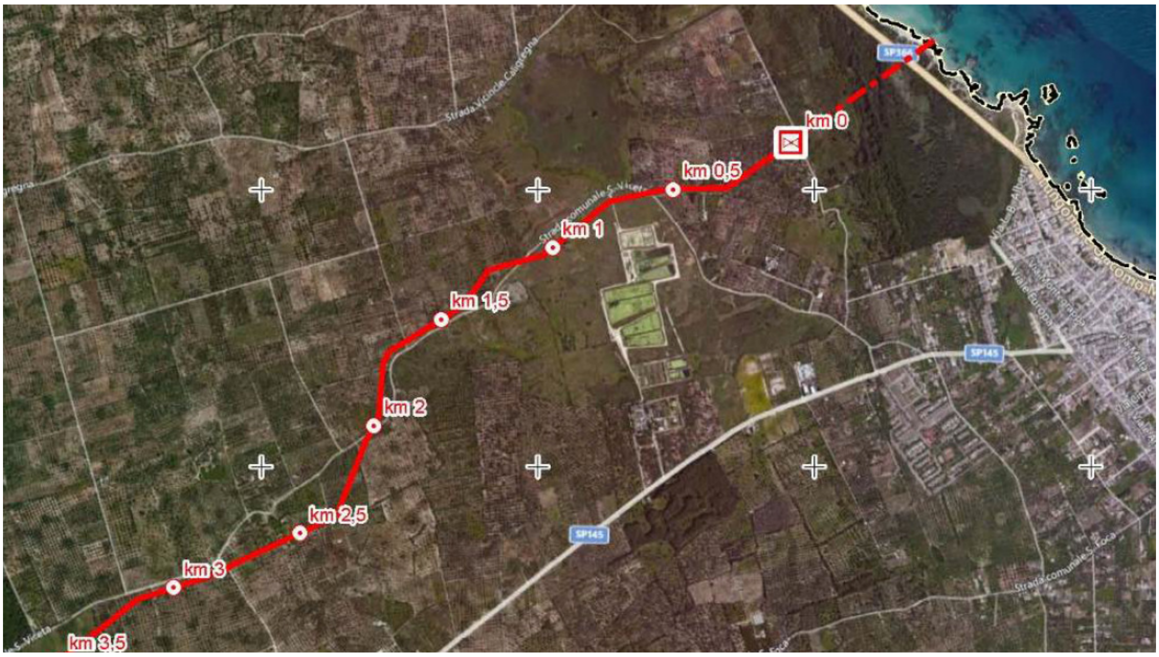



**Figure 3-1: Onshore route, Landfall to KP 3.5** *(source document: IPL00-ENT-100-F-DFO-0001)*

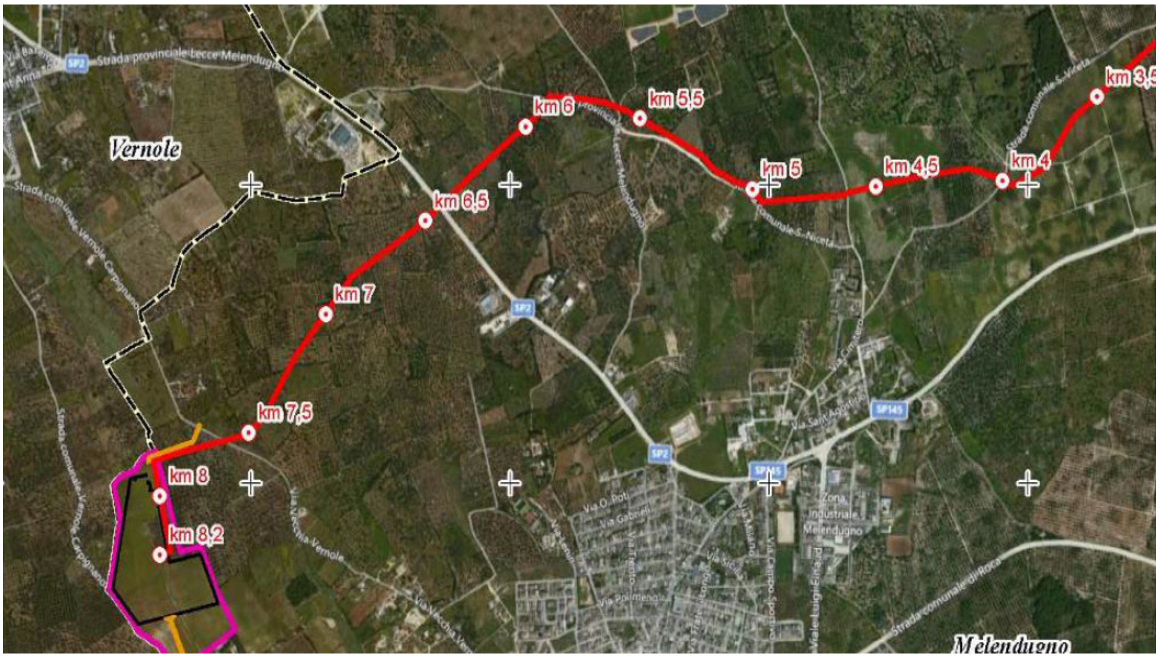



**Figure 3-2: Onshore route, KP 3.5 to PRT** *(source document: IPL00-ENT-100-F-DFO-0001)*

# 4 GEOLOGICAL SETTING

## 4.1 Geomorphology

The landscape of Salento is characterized by a smooth morphology, consisting in a series of undulated plains with various extent and size, generally stretching in NW-SE direction, and characterized by different elevations (Attachment 01). The slopes linking the plains represent mainly fault surfaces or ancient shore escarpements (Sansò et al., 2004).

The most important reliefs are the "Serre", low tabular ridges in the western sector of Salento, NNW – SSE and NW – SE trending, normally cutting Cretaceous–Paleogene limestones and reaching a maximum elevation of 200 m a.s.l. They have a complex structural origin and locally represent portion of an ancient (pre–Miocenic) tropical planation surface (etchplain, Sansò et al., 2004), moulded by dolines with bauxitic residual deposits and by small dome reliefs.

Towards the Adriatic coast the relief is less marked and the tabular ridges are less extended. Along the coast, North of Otranto, a system of rhomb-shaped depressions N-S trending, associated probably with recent tectonic activity, are occupied by lake basins (Sansò et al., 2004).

Due to the extensive presence of carbonate rocks, Salento is particularly affected by karst, which is widespread, from limestones and dolomites of Mesozoic age to the younger units, interesting even Pleistocenic deposits of Gravina Calcarenites.

According to Sansò et al. (2004), four phases of karst development can be recognized:

1) The first one has a Paleogenic age and developed on Mesozoic limestones, producing in a tropical climate a tabular landscape, with intense dissolution processes.

2) The second one took place in Pliocene: only few karst morphologies of this phase can be observed, in Pietra Leccese deposits, maybe due to the intense erosion which affected the landscape.

3) The third one has a Lower-Medium Pleistocene age and was linked to a sea base level lower than the present one. The majority of present karst landforms belong to this phase.

4) The last phase dates Medium-Late Pleistocene. Only few underground landforms can be attributed to this phase, in Gravina Calcarenites.

In Northern Salento underground karst is mainly characterized by hypogeal caves of various sizes, usually with a sub-horizontal development 1) close to tectonic dislocations and/or 2) along the interlayer surfaces of the calcareous formations or 3) as contact-karst between Mesozoic limestones and less soluble Cenozoic formations. These caves can occasionally collapse, hence forming sink-holes, particularly widespread in the coastal zones, along the Ionian and Adriatic sea (Parise et al., 2008).

Typical features of Salento landscape, related to karst, are the absence of a well-developed hydrographic network and the presence of endorheic basins in which enclosed depressions and dolines represent the discharge points of the surface runoff. In these kind of basins no effective drainage net can be defined; runoff is normally dispersed and only locally funnelled in preferred ways around more steep depressions.

In particular the area crossed by the on-shore pipeline is characterised by an almost flat land morphology, with elevations from approx.7 m to 46 m a.s.l...

## 4.2 Geology

Salento belongs to the Apulia foreland region (Apulian platform), formed by a thick, mainly carbonatic sequence of Mesozoic age, covered in transgression by organogenic and/or calcarenitic deposits (Paleogene – Oligocene) and by a thin carbonate – terrigenous succession dating to Quaternary.

The stratigraphic succession, defined by Largaiolli et al. (1969) in the Geological Map of Italy ("F° 214 – Gallipoli; Carta geologica d'Italia a scala 1:100.000") and by Ciaranfi et al. (1988) for the Geological Map of Murge and Salento ("Carta geologica delle Murge e del Salento"), has been recently modified by the detailed studies of Bossio et al. (2005) and Bossio et al. (2006), carried out in the Lecce area and in the coastal region of Salento from Otranto to Santa Maria di Leuca.

A brief description of stratigraphic units of Salento (from the oldest to most recent) is provided below.

**<u>Altamura Limestone (Melissano Limestone and Galatina Dolomite) – Upper Cretaceous</u>**

The Mesozoic carbonate sequence outcrops in the inner part of Salento forming the highest reliefs of the region. The sequence consists of a carbonatic succession with alternating layers of variable thickness of white/grey compact micritic limestones and dolomitic limestones (Bossio et al., 2006). The overall thickness is considerabile, reaching at least 1000 m. Depositional conditions are typical of a wide inner carbonate platform, with occurrence of tidal cyclic successions.

In the "F° 214 – Gallipoli" of the Geological Map of Italy, Largaiolli et al. (1969) established two Mesozoic formations, Galatina Dolomite and Melissano Limestone. Later on Ciaranfi (1988), named Altamura Limestone the Mezozoic sequence of Salento. The denomination has been retained by several authors (Margiotta et al., 2006, Bruno et al., 2008), while Bossio (2006) reprised the first denomination, Melissano Limestone.

**<u>Galatone Formation (Upper Oligocene) and Lecce Formation (Upper Oligocene-Lower Miocene)</u>**

Galatone Formation and Lecce Formation outcrop southwest of Lecce. Galatone Formation is composed by micritic compact grey-white limestones, marls and sandy clay deposits (Bossio et al., 2006). The thickness is low (between 10 m and 70 m). Lecce Formation is characterised by calcarenites varying from white to light brown, with thickness of approximately 60 m.

**<u>Pietra Leccese - Burdigalian-Tortonian (Messinian)</u>**

"Pietra leccese" Formation outcrops extensively around Lecce, and in an area bounded by Struda, Vernole and Acaia. This formation is made up of two slightly different successions (Calò et al., 2005). The bottom is represented by a pale yellow detrital marly biomicrite, with a compact structure. The top of the succession is formed by glauconitic limestones, frequently with a soft and porous structure. The glauconitic limestone is locally named "Piromafo" (Ciaranfi et al., 1988).

The lithologic sequence indicates a littoral to open platform environment. Its thickness reaches the maximum value of about 80 m.

**Andrano Calcarenite - Messinian**

The Andrano Calcarenite occupies wide areas to the East and South-East of Lecce.

This formation represents the regressive term closing the Miocene cycle owing to the emersion of the whole Salento area (Bossio et al., 2006). It is composed by bioclastic limestone, detrital or oolithic limestone, with minor marly limestone and marl. The deposits are composed by soft and porous sediments alternated with compact layers.

The facies association indicates depositional environments near to the inner/outer boundary of the neritic zone in the basal sequence, with evidence of a progressive decrease of the depth in the top succession (Bossio et al., 2006). Its total thickness reaches 50 meters.

**Leuca Formation – Lower Pliocene (Upper Miocene)**

The Leuca Formation, only several meters thick, forms a continuous strip between Andrano Calcarenite and Uggiano la Chiesa Formation. It is constituted by breccias, conglomerates and, subordinately, by glauconitic biomicrites (Bossio et al., 2006). Benthonic assemblages suggest a shallow water marine environment.

**Uggiano la Chiesa Formation – Lower Pliocene**

The Uggiano la Chiesa Formation forms a large strip bounding the coastal line of the Adriatic Sea. It consists of stratified and fossiliferous biodetritical limestones, generally soft, and yellowish calcareous sands, about 50 meters thick. The basis of the succession is characterized by a conglomerate layer, constituted by phosphatic cobblestones in a light coloured calcareous-phosphatic matrix. The lower sequence is normally fine grained, and locally has a marly composition, while the upper sequence is represented by medium to coarse grained calcareous sediments. In the Lecce area, the depositional environment indicates the outer or inner neritic zone (Bossio et al., 2006).

**Gravina Calcarenite (Salento Calcarenite) – Plio-Pleistocene**

The Gravina Calcarenite outcrop eastwards from Calimera and westwards from Vernole, up to Borgagne. This unit is characterized by a considerable lithological variability: in fact it includes medium to fine sized, little coherent marly calcarenite; coarse fossiliferous calcarenite; coarse calcareous sand; silty sand or sandy silt more or less cemented; in general calcarenite is yellowish or greyish in colour, while the other lithotypes can also be covered with a yellow or reddish surface crust. The depositional environment is littoral. The Gravina Calcarenite corresponds to the calcarenitic facies of Plio – Pleistocene age belonging to the Salento Calcarenite of the "F° 214 – Gallipoli of the Carta geologica d'Italia".

**Subappennine Clay - Pleistocene**

Subappennine Clay does not outcrop in the coastal area, but are recognized underground, in several stratigraphic logs of wells. The unit is composed principally by stratified clay and marly-silty clay, with rare sand intercalations (Ciaranfi et al., 1988). The thickness can reach 250 m in the western part of Salento.

**Recent Continental Deposits**

Along the coastline, recent continental deposits comprise clayey and silty sediments with peat layers, of lagoon or marsh environment, and eolian sands forming coastal dunes.

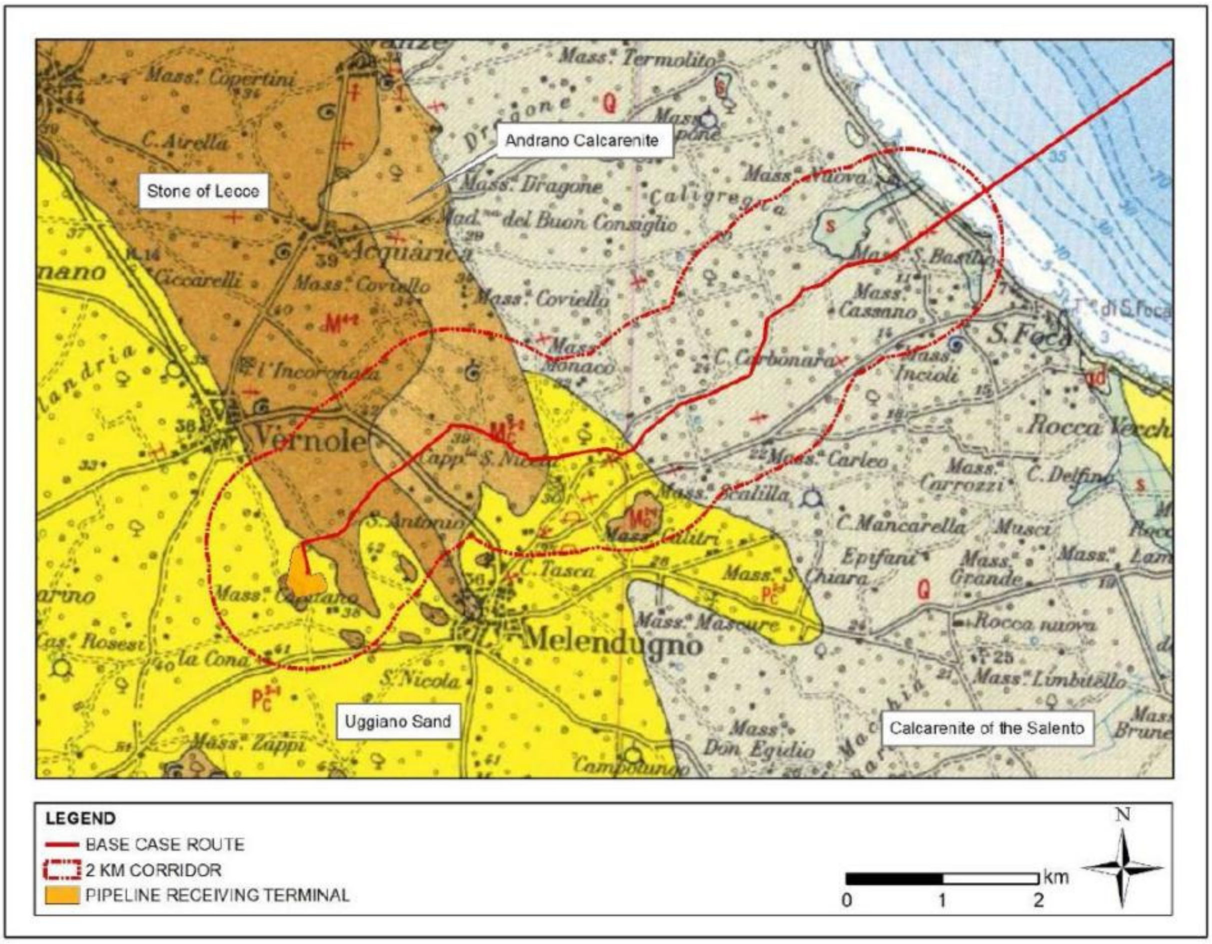



**Figure 4-1: Geological Map of Italy ("F° 214 – Gallipoli; Carta geologica d'Italia a scala 1:100.000")**

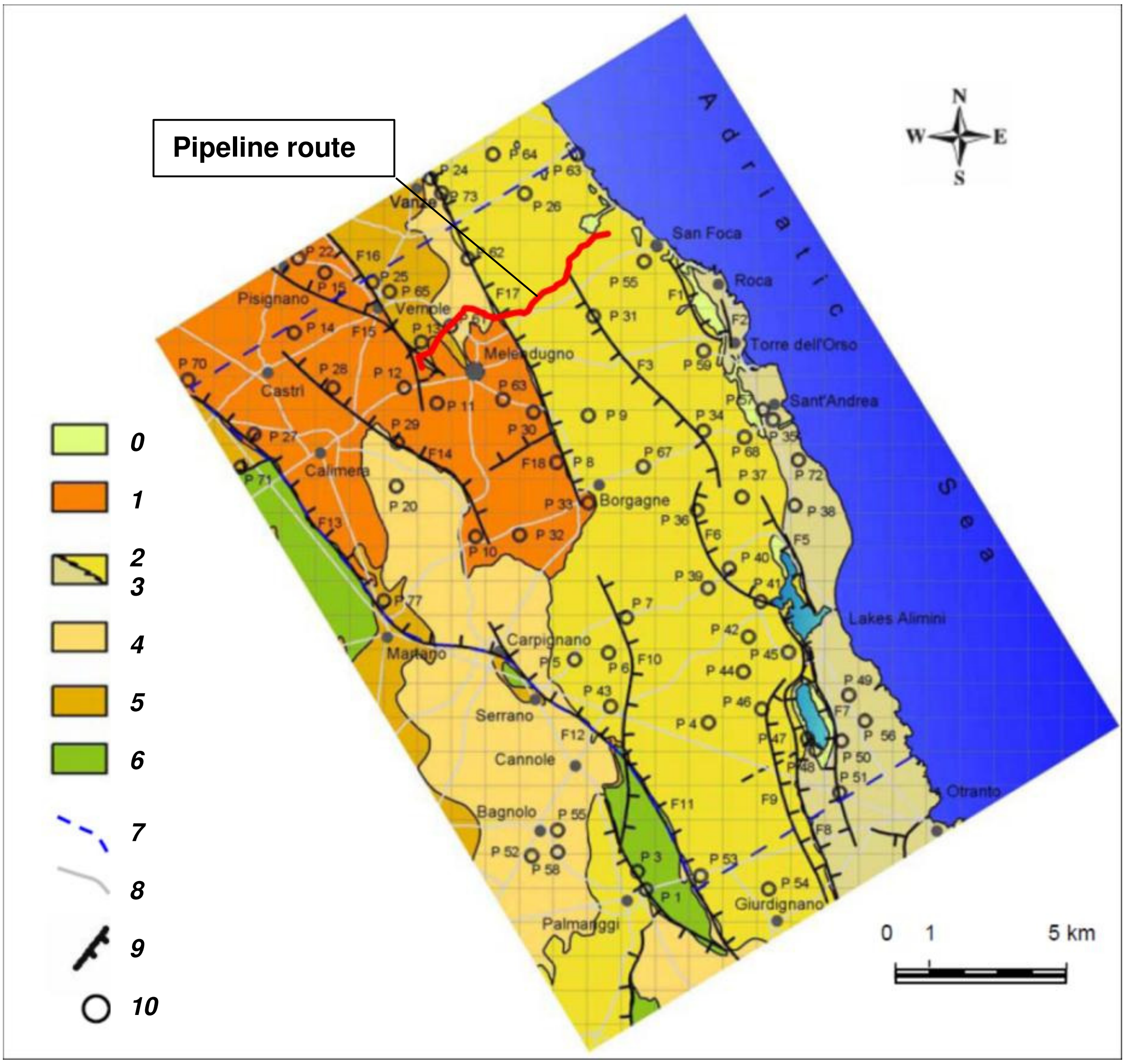



**Figure 4-2: Geological map of the area surrounding the pipeline route (red line). After Bruno et al., 2008, modified. LEGEND: (0) Alluvial sand and loam (Recent); (1) Coarse-grained sandstone "Calcareniti di Gravina" formation (Plio-Plesitocene); (2) Detrital sand with calcarenite layers interbedded, (3) Glauconitic calcilutite "Sabbie di Uggiano" formation (Pliocene); (4) Medium-grained calcarenite with marl limestone layers interbedded "Calcarenite di Andrano" formation (Upp. Miocene); (5) Marl fine-grained calcarenite (Upp.Miocene) "Pietra Leccese" formation, (6) Limestone and dolomitic limestone (Upp. Cretaceus) "Altamura" Formation; (7) investigated area boundary; (8) principal roads; (9) probably faults; (10) drill logs.**

The Pipeline route crosses an area of the "Sabbie di Uggiano" formation, the "Calcarenite di Andrano" formation, the Pietra Leccese" formation and the "Calcareniti di Gravina" formation.

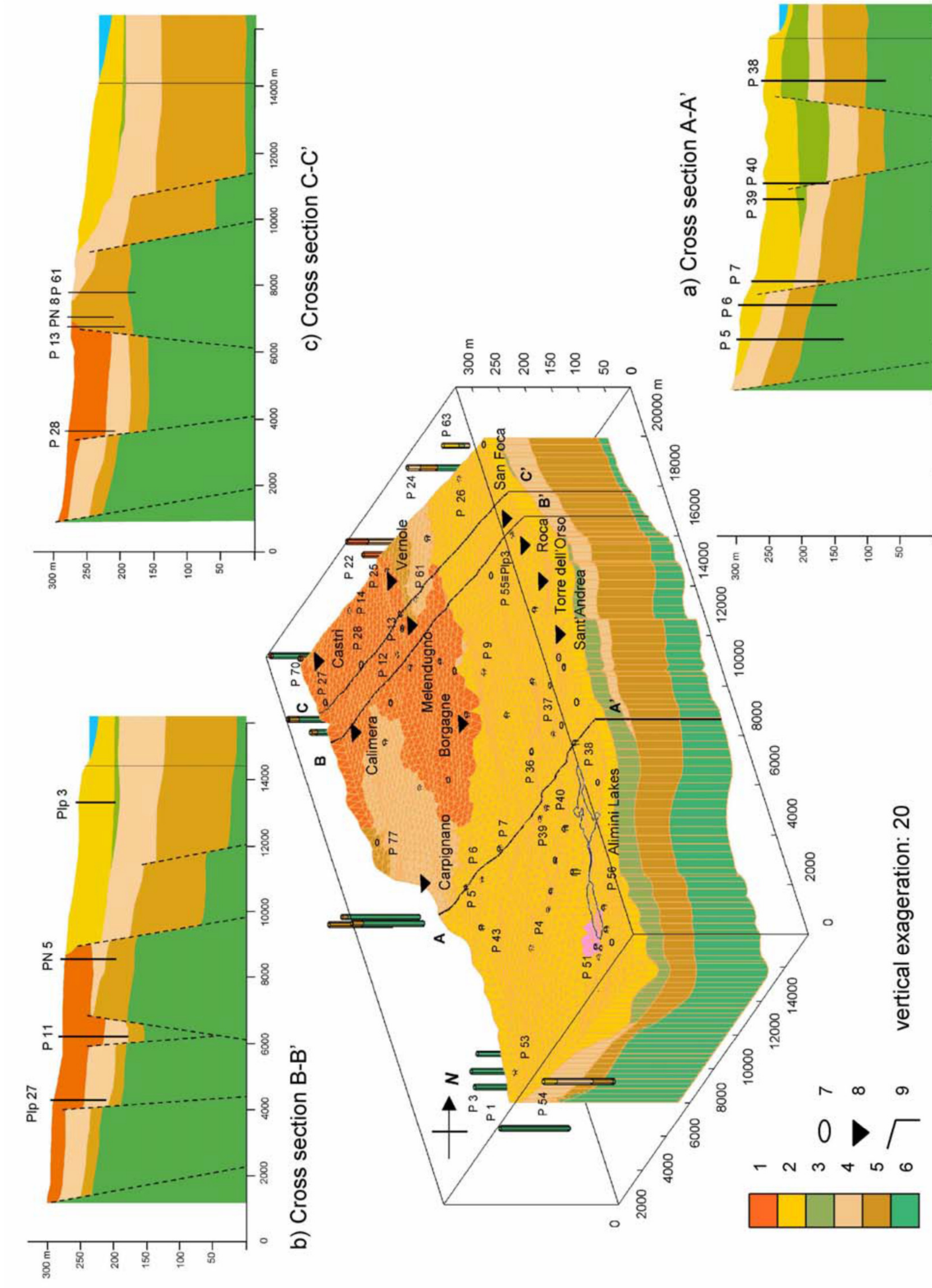



**Figure 4-3: Geolithologic 3D model with extrapolated cross section: (1) coarse-grained sandstone, (2) sand with sandstone layers interbedded, (3) sandy-clay and clay marl, (4) medium-grained calcarenite with marly limestone layers interbedded, (5) marly fine-grained sandstone, (6) fractured limestone and dolomitic limestone, (7) drill log position, (8) principal town location, (9) cross section line.**

## 4.3 Structural settings

The Apulia foreland is deformed by a broad antiformal fold with a WNW – ESE trend, extending from the Bradanic basin to the Adriatic sea (Doglioni et al., 1996).

The structure of the southern Salento is dissected by NW – SE trending normal faults, of variable ages (Late Cretaceous to Pleistocene), forming a series of structural reliefs (horst) and tectonic depressions (graben) extended along the axial lines with NW-SE direction.

On a regional scale, the Puglia antiform shows the larger down faulted blocks dipping towards the Bradanic trough and the Adriatic sea (Doglioni et al., 1996). Owing to this structural setting the Mesozoic formation outcrop in the inner part of the region.

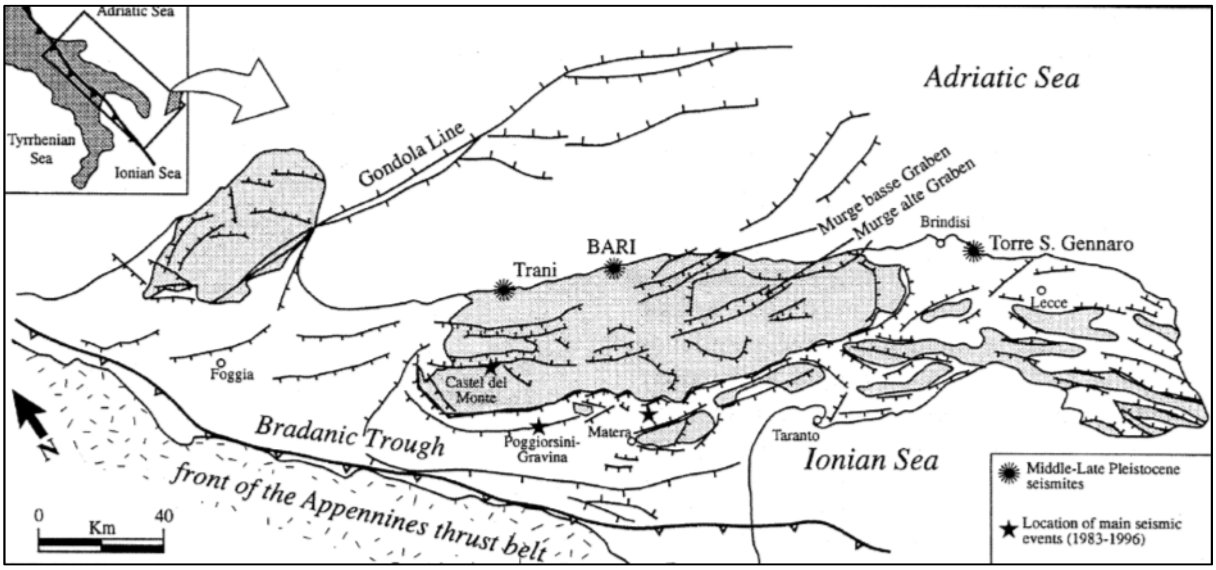



**Figure 4-4: structural sketch of the Apulian foreland: localization of seismites and main instrumental seismic shocks (Quaternary tectonic activity of the Murge Area – Apulian foreland – Southern Italy. Pieri et alii, 1997)**

Locally, in the study area, the structural setting is roughly monoclinal, with slight immersion towards the Ionian coastline (Ciaranfi et al., 1988).

## 4.4 Seismic hazard

### 4.4.1 Italian Seismic Classification

According to Italian Legislation (Legislative Decree no 122 of 1998 and Decree of the President of the Republic no. 380 of 2001 - "Testo Unico delle Norme per l'Edilizia"), the entire national territory has been classified as it follows.

- Zone 1 - It the most dangerous area, where major earthquakes may occur.
- Zone 2 - Municipalities in this area may be affected by quite strong earthquakes.
- Zone 3 - Municipalities in this area may be subject to modest shocks.
- Zone 4 - the least dangerous: municipalities of this area have a low probability of seismic damages.

The Italia Government has filled out a list of municipalities with the zone each of them belongs to, with a decreasing standard of dangerousness.

De facto, there is no such thing as an "unclassified" area, that becomes zone 4 here, within which the Regions have the power of making the antiseismic planning mandatory. Moreover, each zone has a value of the seismic action useful for the above planning, expressed in terms of maximum acceleration in rock (zone 1=0,35 g, zone 2=0,25 g. zone 3=0,15 g, zone 4=0,05 g).

A new study, attached to the OPCM no. 3519/06, supplied the Regions with an updated tool for territorial classification, introducing intervals of acceleration (ag), with a probability of exceeding the threshold equal to 10% in 50 years, to be assigned to the 4 seismic areas.

**Table 4-1: Division of the seismic areas according to the acceleration of peak on rigid ground (ag) (OPCM 3519/06)**

| Seismic zone | Acceleration with probability of exceeding equal to 10% in 50 years (ag) |
|---|---|
| 1 | ag >0,25 |
| 2 | 0,15 <ag≤ 0,25 |
| 3 | 0,05 <ag≤ 0,15 |
| 4 | ag ≤ 0,05 |

Based on addresses and criteria established at national level, some Regions have classified the territory in four zones, as described in the table, and some others have classified it by adopting three zones, and introducing, in some cases, also subzones, to better adapt regulations to seismicity features.

Details and meanings of zonation according to each Region are contained in the regional regulations. Regardless of the regional choice, each zone or subzone has a core dangerousness value, expressed in terms of maximum acceleration on rigid ground (ag). This value does not influence planning.

Current Technical Regulations for Buildings (Ministerial Decree of 14 January 2008), in fact, have indeed modified the role that seismic classification had for planning purposes: for each zone – and thus municipal territory – a value of peak acceleration, and consequently a spectrum of elastic response, was previously supplied to calculate seismic actions. Starting from July 1st 2009, 2008 Technical Regulations for Buildings came into force: each building has its own acceleration, according to geographical coordinates of the project area and to the nominal design life of a building: the degree of core dangerousness, then, can be defined for each point of the national territory, within an area of 5 sq. metres, regardless of local administrative borders. Seismic classification (which seismic zone a municipality belongs to) is thus useful only for planning management and territorial control by relevant boards (Region, Genio, etc.).

The Salento sub-region is classified as “Zone 4”. Municipalities of this area have a low probability of seismic damages.”

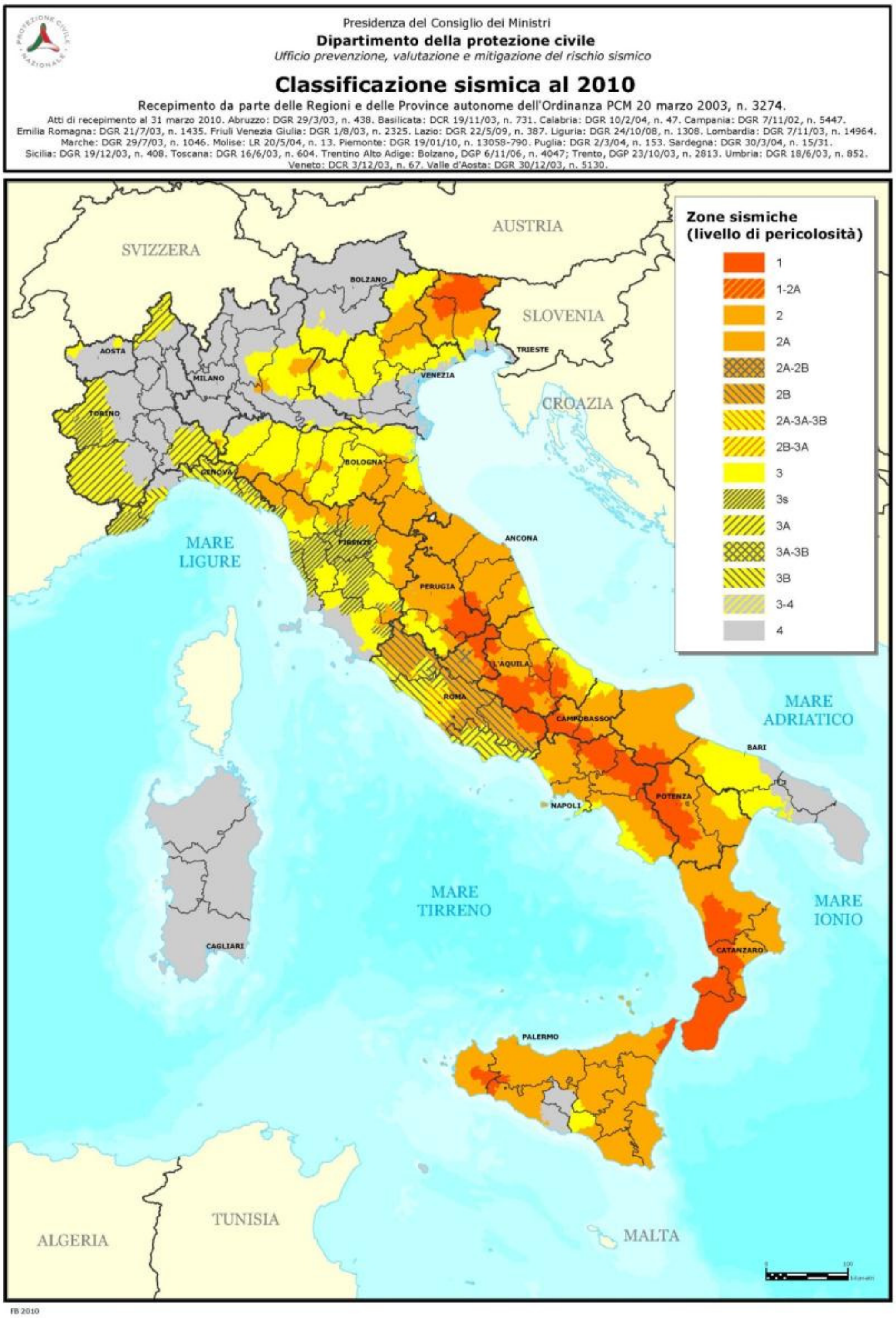
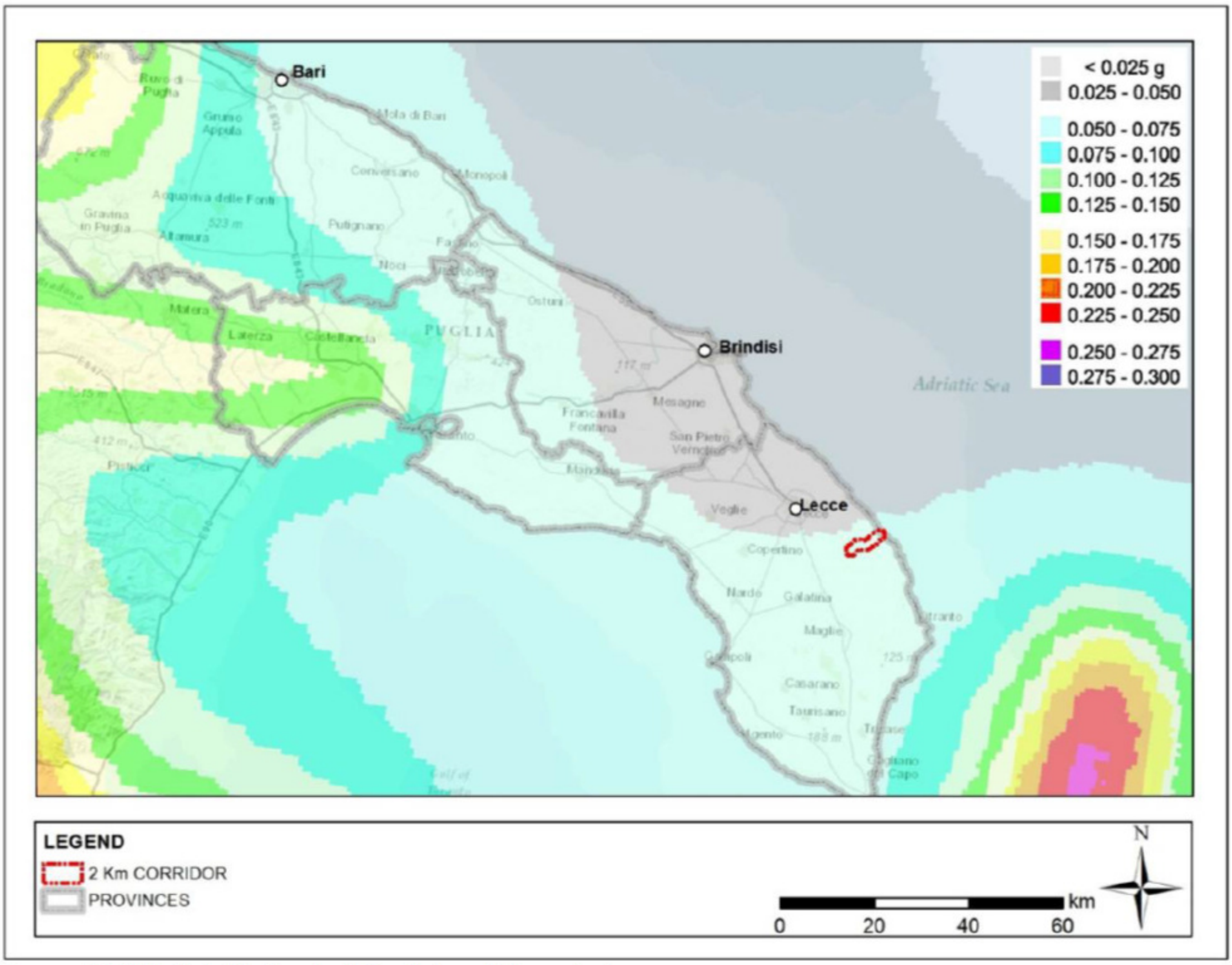



**Figure 4-5: Italian Seismic classification**

Otherwise, the seismic regional law (DGR of Apulia Region no 153 2nd March 2004 – “Individuazione delle zone sismiche del territorio regionale e delle tipologie di edifici ed opere strategici e rilevanti: approvazione del programma temporale e delle indicazioni per le verifiche tecniche da effettuarsi sugli stessi") requires that the anti-seismic design of new buildings and infrastructures defined as strategic and relevant for civil protection and classified in Zone 4 (such as PRT), must be done according to technical standards defined for Zone 3.

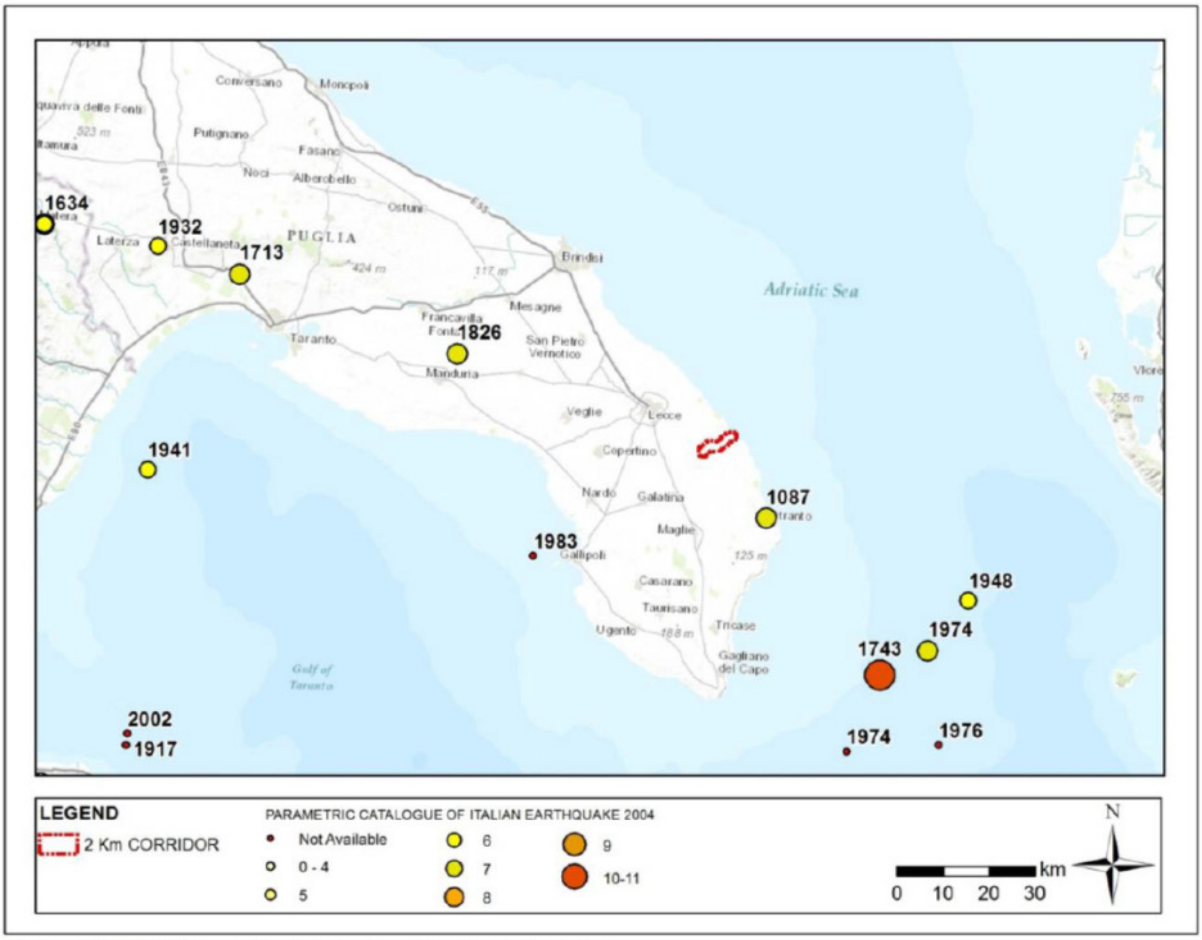
Few earthquakes have been recorded in Salento throughout history,. This area can be catalogued as having low seismic activity based on the epicentre distribution of the historical events and/or the current Italian map of seismic hazard (Ordinanza PCM, 2006), where southern Apulia is characterized by 10% probability to exceed 0.050-0.075 g peak ground acceleration (PGA) value in 50 years (Figure 4-6).




*Source: INGV Istituto Nazionale di Geofisica e Vulcanologia*

**Figure 4-6: Seismic Hazard**

Figure 4-7 shows the registered seismic events around the Study Area (271 B.C – 2002 A.D.). The epicentral intensity (MCS, Mercalli Scale) was reported with the aim of using this parameter as a homogeneous system to measuring tectonic activity.




*Source: CPTI04 Parametric Catalogue of Italian Earthquake*

**Figure 4-7: Major Earthquakes Recorded (Scale MCS)**

The 1743 earthquake is parameterized by Working Group CPTI (2004; hereafter CPTI04) with an epicentral intensity Io=IX-X MCS, and an average magnitude Maw=6.9, its epicentre being located offshore of the southeastern Salento coast. CPTI04 report other earthquakes with the epicentres in Salento:

- 1826, with light damage in Manduria and Crispiano
- 1087, with damage in the Otranto area

In conclusion, on the basis of the distribution of historical earthquakes in the area of interest and of the seismogenic characteristics of the region, the territory through which the planned route is to run presents a very low seismogenic index, both as regards the frequency of the events and as regards their magnitude.

With regard to seismic hazard, the maximum expected horizontal ground acceleration values at bedrock level for planned pipeline, defined according to the recent technical rules (NTC, 2008), are particularly low.

## 4.5 Hydrology

### 4.5.1 Surface Water

In the Salento Peninsula, the surface hydrography is represented, rather than by water courses in the real sense of the term, by flow lines into which the rainwater is channelled and which affect the topographic lowest areas, where lithological outcrops are predominantly sandy, sandy-clayey and calcarenitic, externally delimited by calcareous ridges.

These lines drain rainwater by conveying the runoff to doline-shaped hollows (so-called "cupe") and/or to dolines and swallow-holes, areas often subject to flooding events during heavy rainfall from which it is dispersed in the carsic subsoil. The absence of a well-developed surface drainage net is a characteristic feature of karst landscape.

The planned route does not cross any permanent or seasonal watercourse. In the proximity of the landfall of the on-shore pipeline, only two seasonal streams are mapped. Approximately 530 m North, a streams runs parallel to the route, connecting the wetland to the sea (Figure 4-8). The other stream has its outlet about 350 m south.

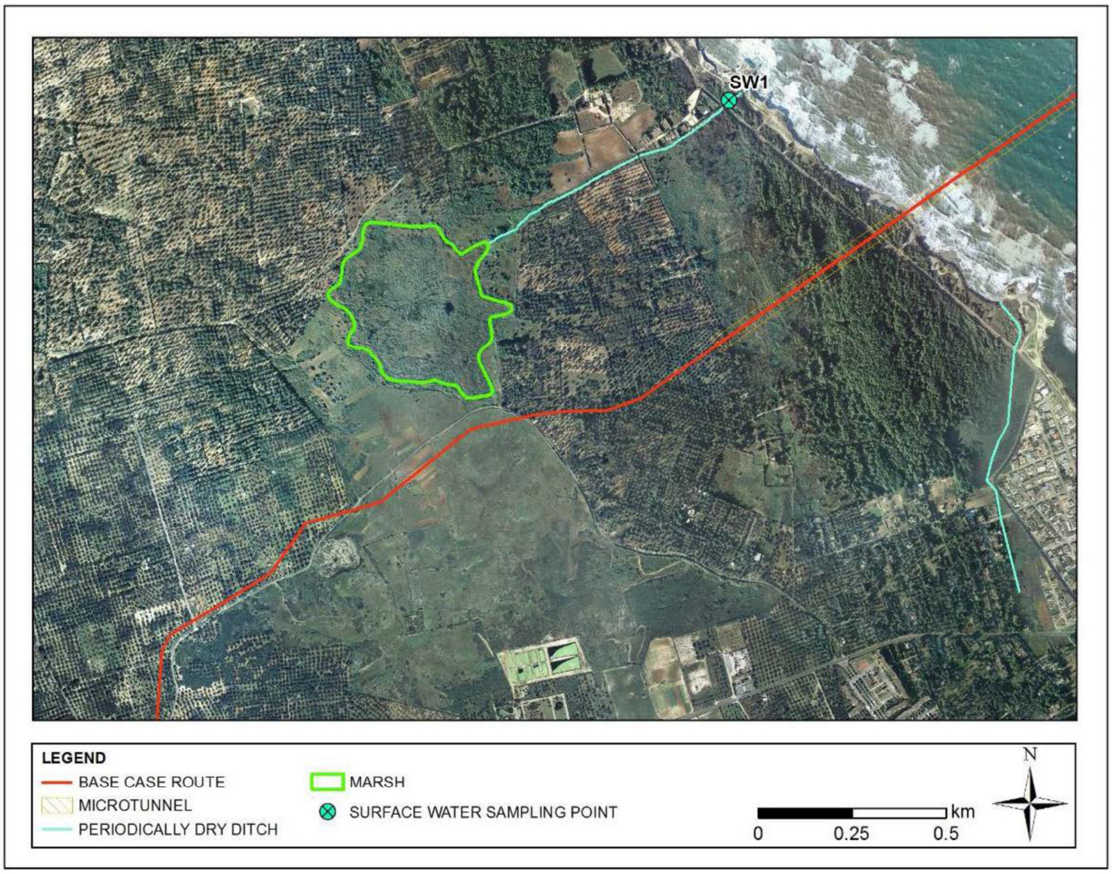



**Figure 4-8: Surface water** *Source: ERM (November 2011)*

From a hydraulic and hydrogeological point of view the route runs near two endorheic areas under investigation by Basin Authority, at the Kp 4.5 and 5.5 (according to the official communication n° AO Prot 8/10/2012 8.50 0011854 of Basin Authority). Although no restrictions are foreseen in these areas by PAI (Piano di Assetto Idrogeologico, Hydrogeologic Setting Plan), Basin Authority requires that pipeline-laying shall not adversely affect the hydraulic regime of the areas. Additionally any sediment build-up, resulting from exceptional weather events, shall not cause pipeline dysfunction and / or inefficiencies. These aspects are verified through the Hydraulic Compatibility Study of the Pipeline IPL00-URS-000-Q-TRS-0001.

### 4.5.2 Groundwater

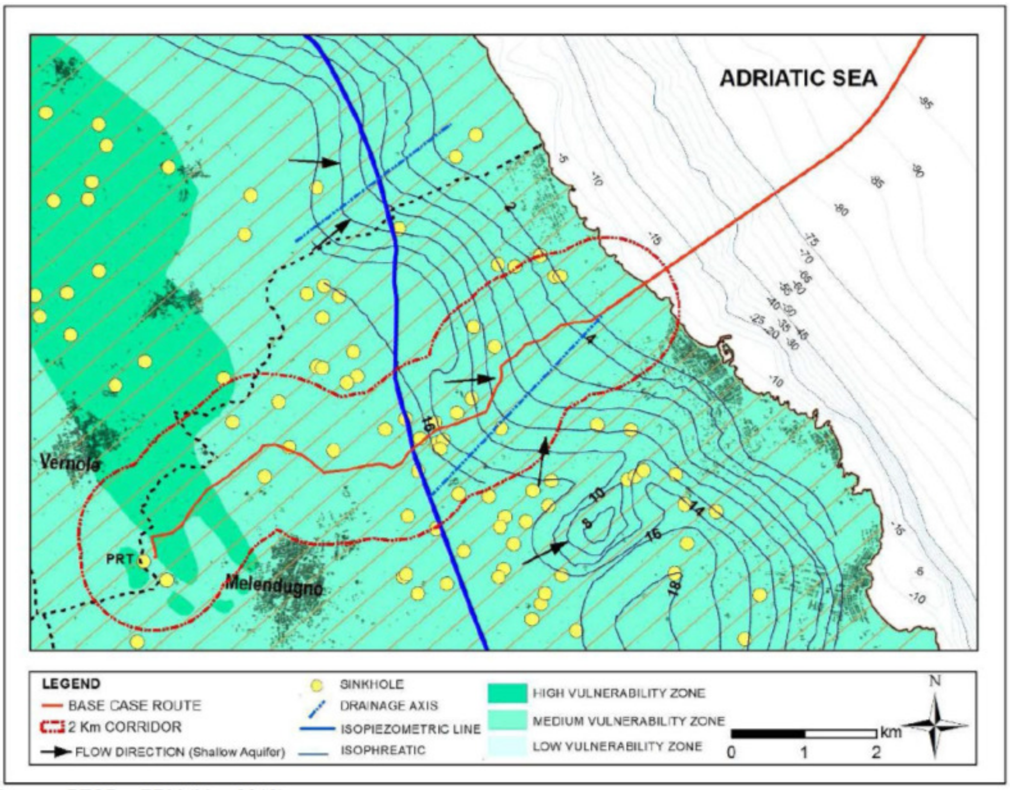
Apulia represents a complex hydro-geological environment. The Salento sub region is characterised by two aquifers: the first one, shallower, is made up of Mio-Plio-Pleistocene sediments holding one or more bodies of groundwater. The geometry of the second one is often difficult to determine, since the sediments lie in limited intervals of permeable rock in a more general context of impermeable deposits. The second, deeper, aquifer is made up of Mesozoic carbonatic formations. In details:

- the shallow aquifer is located in the Calcareniti del Salento and Sabbie di Uggiano formations. Its charge is due almost exclusively to the precipitation falling on their outcrops in the territory. It shows a degree of permeability related to the percentage content within the sand of silt and / or silty-clay. The storage capacity is generally not high. The water table is subject to seasonal variations level;
- the subappenninic clays form an aquitard that separates the shallow aquifer from a semi-confined aquifer located in the Calcareniti di Andrano. It is connected with the shallow aquifer;
- Pietra Leccese represents an aquiclude that separates the multilevel shallow aquifers from the deeper aquifer located in the Calcare di Altamura formation; the deeper groundwater is thus confined in these Cretaceous deposits by the overlying Miocene sediments (generally impermeable).

The shallow aquifer and the semi-confined aquifer belong to the system called shallow multi-level aquifer. The project route crosses the morphologically depressed areas occupied by Plio-Pleistocene terrains: here the multilevel shallow aquifer can be found. Particularly, in the initial section (within the first kilometer), the water table is found at a depth of approximately 6 m (July 2013); groundwater depth gradually increases from the coast towards the inland. This does not exclude the presence of perched/discontinuous groundwater bodies contained in the calcarenitic terrains and sustained by marl levels of the calcarenitic-marly formations.

The following Figure 4-9shows PRT, pipeline route and microtunnel overlaid to the hydrogeological map provided by PTCP (Provincial Coordination Territorial Planning) of Lecce. It highlights:

- the groundwater contour lines: in the shallow aquifer contour lines gradually decrease from 16 m to 2 m above sea level);
- the groundwater flow direction of the shallow aquifer, as indicated by the arrows (Figure 4-9);
- the presence of drainage axes (SW-NE) that characterise the shallow aquifer;
- the vulnerability degree of the aquifer, linked to primary and secondary permeability, due to silt and/or clay content within the sands, and the grade of cementation of the Calcareniti.




*Source: PTCP – ERM (May 2013)*

**Figure 4-9: Pipeline route and hydrogeological vulnerability map (provided by PTCP (Provincial Coordination Territorial Planning of Lecce)**

# 5 FIELDWORK DESCRIPTION

Geotechnical and geophysical investigation were carried out as per documents from 3 to 6 listed in Chapter 1.

The following tasks were carried out:

- unexploded ordnance (hereinafter referred to as UXO) survey;
- geophysical investigation (ERT imaging, seismic refraction, Multichannel Analysis Surface Waves),
- geological and geotechnical investigation (soil borings, sample collections and in-situ testing),
- archaeological surveillance during soil borings;
- topographical survey of the investigated points,
- laboratory testing on the collected samples.

In addition, top-soil samples were collected to be analyzed in chemical laboratory in order to derive the composition of the soil mixture (1.7.120, Bill of Quantities 19/09/2012).

UXO survey was performed in July 2014 on an area 25 $m^2$ wide surrounding each borehole and down to a depth of 5 m below ground level by a company accredited by the Ministry of Defense and gave no evidence of unexploded ordnances on and below ground. Similarly, the archaeological surveillance gave no evidence of anthropic artefacts both during the above mentioned UXO survey and in cores during soil borings.

More details on the investigations carried out are described in the following chapters.

## 5.1 Geophysical investigation

On June 2013 URS performed a first geophysical investigation campaign. An overview map with the location of the site activities is provided in IAL00-URS-000-Q-TRG-0001_00—Geophysical Investigation Italy - Annex A - Appendix E.

The purpose of the survey was to map the subsurface structure of PRT areas and along the pipeline route , in particular the depth of base-rock, to allow planning pipeline installation works, to possibly identify potential karst formations and to highlight areas were further more detailed investigation could be needed.

A further geophysical investigation was commissioned to URS, which was performed from March to April 2015.

Both of the two geophysical campaigns were conducted combining two investigation techniques, Resistivity Imaging and Seismic Refraction, commonly used in combination, because measuring both the electrical and mechanical properties of the subsoil allows cross-confirmation of results, improves accuracy and highlights areas of differences which may require further investigation.

Resistivity Imaging, also known as Electrical Resistivity Tomography (ERT), is based upon the measurement of changes in electrical resistance produced by variations in materials within the ground. These variations can be caused by factors such as the material changes (e.g. basalt compared to sandstone) or different local environmental conditions such as water content and compaction.

Electrical resistivity is measured by applying an alternating current into the ground, and measuring the electrical potential created by the current. Four electrodes are used to for each measurement, two of which are used to pass the current through the ground, and two to measure the resulting potential. The ratio of current to potential gives the electrical resistance of the ground.

Seismic refraction surveying relies on how seismic waves propagate through the ground and interact with areas of different density within the subsurface. Seismic energy is generated from the surface by an impact or explosion (known as a "shot") and the arrival times of seismic waves are recorded by geophones arranged in a linear array along the surface.

The velocity of a seismic wave is linked to the density and elastic module of the material through which it travels. A seismic wave encountering a sufficient change in velocity will be refracted at an angle determined by the magnitude of the change. As the seismic velocity of the lower material is generally higher than that of the upper material, there will be a point in the geophone array where the seismic energy travelled along the boundary overtakes the seismic energy propagating through the upper material and becomes the first arrival at the geophone. Analysis of these first arrivals allows a seismic velocity model to be created.

A total of 32 ERT profiles were realized during the 2013 survey along the pipeline route; the total length surveyed was 7.41 km. The electrode spacing for ERT survey was 3 m. GPS coordinates were collected at the beginning and at the end of each profile line. Topographic elevations along the profiles were derived from official DEM, publicly available in the Puglia Regional Administration website.

A total of 112 seismic refraction spreads were collected along the pipeline route in the 2013 campaign, for an overall length of 7.36 km. Seismic refraction profiles were typically 69 m long. The geophone spacing for the refraction survey was 2-3 m throughout and was chosen on the basis of available space for laying the spreads. An effective depth of investigation of approximately 16m was achieved across the survey area. As done for ERT, GPS coordinates were collected at the beginning and at the end of each profile line and topographic elevations along the profiles were derived from official DEM, publicly available in the Puglia Regional Administration website.

In 2015, a total amount of over 6.5 km geophysical investigation was performed along pipeline route (one of the profiles, PR06, being partially within PRT area); the survey was carried out with the same method adopted in the 2013 campaign (electrode/geophone spacing, roll along technique, array length, etc…).

Finally, a seismic investigation using the MASW technique has been performed in the vicinity of Piezo2 investigation point. MASW technique, using similar equipment and setup to those used for seismic refraction, allows to record information about the trend of shear wave velocity (Vs) vs. the depth. Being Vs velocity correlated with soil engineering properties, this method allows to get information on soil categories which are needed for design, particularly for seismic design. For details please refer to chapter 6.4.1.

The following is a summary table listing the geophysical investigation performed.

**Tab. 5-1: summary of the performed geophysical investigation**

| TECHNIQUE | CAMPAIGN | NO. OF PROFILES | OVERALL LENGTH |
|---|---|---|---|
| Electrical Resistivity Tomography (ERT) | 2013 | 32 | 7410 m |
| Seismic refraction | 2013 | 112 | 7360 m |
| Electrical Resistivity Tomography (ERT) | 2015 | 6 | 6500 m |
| Seismic refraction | 2015 | 6 | 6500 m |
| MASW | 2015 | 1 | 46 m |

The final results for the Resistivity and Seismic Refraction Survey are presented in Attachment 4 and Attachment 5 and in Annexes A and B of IAL00-URS-000-Q-TRG-0001_00—Geophysical Investigation Italy.

## 5.2 Geotechnical investigation

Geotechnical investigation along the pipeline route was aimed to:

- define the local stratigraphy, geological and hydrogeological setting
- collect samples to be analyzed in laboratory and perform in-situ testing for geotechnical characterization of subsoil along the pipeline route.

Geotechnical investigation initially started on April 2015, with the drilling of BH3B investigation point, after the completion of PRT soil investigation and ended on June 2015.

18 soil boring were drilled along the pipeline route to a depth of 10 m, for a total length of 180 m. Boring locations are shown in Attachment 2. Boring were drilled using continuous coring technique with a core diameter of 101 mm, as per tender documentation listed from 3 to 6 in chapter 1.

During the drilling of those 18 boreholes, 16 SPTs *(Standard Penetration Test, tests carried out on site to measure the number of strokes needed to drive a standardized sampler into the ground under the blows of a hammer with a weight of 63.5 kg and a height of 76 cm; the extent of penetration make it possible to obtain, through the correlations, geotechnical parameters such as the friction angle, relative density, cohesion, etc.)* were performed and 20 samples were collected for the grain size analysis, natural unit weight, water content, Atterberg limits, water permeability by oedometer.

**Tab. 5-2: summary of geotechnical investigation**

| METHOD | AMOUNT |
|---|---|
| boreholes | 18 |
| Standard Penetration Test | 16 |
| Collected samples | 20 |

# 6 ANALYSIS OF THE RESULTS

In this chapter the results of geophysical and geotechnical investigation are presented, discussed and interpreted in order to derive geophysical, geological and geotechnical characterization of the soil and subsoil along the pipeline route.

## 6.1 Geophysical investigation

### 6.1.1 2013 campaign

Profiles along the pipeline are presented East to West, allowing a continuous chainage from the the coast to PRT.

The Pipeline Route can be divided into several main areas:

• from chainage 0 to 3985 m: Materials appear to be bedded horizontally. Thin (1-2m though up to 5m locally) soils overlying a high resistance, low velocity rock layer (layer 2 above) of up to 5m, over a low resistance, medium velocity aquifer (layer 3 above).

• from chainage 3985 m to approx. 4620 m. Materials appear to be bedded horizontally, with a high resistance, low velocity layer (layer 2 above) of up to 15m containing localised pockets of conductive materials, underlain a conductive layer medium velocity layer (layer 3 above).

• from chainage 4620 m to 6150 m. Layers appear to dip to the East, with layer 2 pinching out at around 4800 m and layer 3 pinching out at 5200 m. A further high resistance layer is evident to 5410 m where there is a distinct increase in both seismic velocity and resistivity of the near surface layers, suggesting layers 4 and 5 outcropping.

• from chainage 6150 m to 6260 m. This relates to a large sink hole. Depth is not well resolved in the resistivity data, but the seismic data suggests this to be around 10m below the current surface.

• from chainage 6260 m to 8020 m. No obvious dip evident in the profiles, with layers 4 and 5 outcropping. Velocity variations appear to suggest an uneven profile to this harder rock filled with lower velocity materials, suggesting either karsts filled with later deposits or an uneven weathering pattern within this layer.

• from chainage 8020 m to 8720 m. Layers appear to dip to the West, with layer 4 being overlain by layer 3 from 8550 m.

The geology of these areas is summarised in the following Table 6-1.

**Table 6-1: Interpreted layers along pipeline route.**

| Chainage [m] From | To | Kp. | Dip | Layer | Rock Layers | Thickness | Comments |
|---|---|---|---|---|---|---|---|
| 620 | 3985 | 0.0 - 3.4 | Level | 1 | Soil/Boulders | 0-3m | Up to 5m locally |
| | | | | 2 | Highly Weathered Rock | 0-5m | High resistance, low velocity layer thickens locally around outcrop from 1600m to 1700m. |
| | | | | 3 | Highly Weathered Aquifer | Typically 8m+ | Low resistance, medium velocity material likely to represent a weathered rock aquifer. Resistance values suggest some degree of salinity in the aquifer. |
| 3985 | 4620 | 3.4 - 4.0 | Level | 1 | Soil/Boulders | 0-2m | |
| | | | | 2 | Highly Weathered Rock | 10-15m | Discontinuity at 3985 with an increase in thickness of the upper rock layer/ step down in the water table. This could relate to a change in the underlying geology. |
| | | | | 3 | Highly Weathered Aquifer | Not Resolved | Low resistance, medium velocity material likely to represent a weathered rock aquifer. Resistance values suggest some degree of salinity in the aquifer. |
| 4620 | 6100 | 4.0 - 5.5 | East | 1 | Soil/Boulders | 1-3m | Up to 5m locally |
| | | | | 2 | Highly Weathered Rock | 0-5m | High resistance, low velocity layer pinches out at around 4800m. |
| | | | | 3 | Highly Weathered Aquifer | Not Resolved | Low resistance, medium velocity material likely to represent a weathered rock aquifer outcrops from around 4800m , pinching out at around 5210m. Resistance values suggest some degree of salinity in the aquifer. |
| | | | | 4 | Weathered Rock | Not Resolved | High resistance, higher velocity rock outcrops from 5210m to 5430m |
| | | | | 5 | Hard Rock | Not Resolved | Very high resistance layer containing pockets of very high resistance rock, outcropping from 5430m and appearing to pinch out at around 6100 |
| | | | | 6 | Conductive layer | Not Resolved | Evident beneath previous layer from 5930m dipping to the east. This layer does not appear to outcrop |
| 6100 | 6175 | 5.5 - 5.6 | Not Resolved | 5 | Hard Rock | Not Resolved | Approx. 5m of surface layer underlain by very high resistance feature. No change in seismic velocity suggests this is not an open void. |
| 6175 | 6260 | 5.60 | Not Resolved | 1 | Soil/Boulders | Not Resolved | Resistivity and seismic data suggest a sink hole, which corresponds with a localised dip in topography. |
| 6260 | 7900 | 5.6 - 7.3 | NA | 1 | Soil/Boulders | 0-3m | Up to 5m locally |
| | | | | 5 | Hard Rock | Not Resolved | Very high resistance layer containing pockets of very high resistance rock, outcropping locally. |
| 7900 | 8720 | 7.3 - 8.1 | West | 1 | Soil/Boulders | 0-3m | Up to 5m locally |
| | | | | 5 | Hard Rock | Not Resolved | High resistance Layer containing pockets of very high resistance rock, outcropping locally. Rock outcrops from 7900 to 8580m after which it is overlain by a more conductive layer. |
| | | | | 4 | Weathered Rock (Pliocene Limestone?) | Not Resolved | High resistance, higher velocity rock outcrops from 8580m |

A number of discrete features are evident along the profiles surveyed. One can be confirmed as a large sink hole, but the majority of others should be seen as potential localised hazards, including discontinuities in the rock such as faults, karsts, filled channels and localised dips in the strata. The majority of these features have been tabulated below, but it cannot be guaranteed that all features will have been identified or that all the features identified are accurately defined.

**Table 6-2: Discrete features along the route**

| Resistivity Chainage | Resistivity Profile | Comments |
|---|---|---|
| 1400-1540 | 6-6B | Localised thickening of weathered rock |
| 3920 | 13 | Discontinuity in geology may suggest fault. |
| 3985 | 13 | Apparent step down in geology suggests fault. |
| 4210 | 13 | Localised low resistance feature within the high resistance rock suggests potential water filled cavity. Seismic data shows high velocity feature beneath this. |
| 4290 | 13 | Localised low resistance feature between the high resistance layers suggests potential break in the rock. |
| 4780 | 18 | End of the outcrop of upper high resistance low velocity material |
| 5035 | 18 | Potential localised discontinuity within rock suggested by resistivity data. Seismic data suggests a higher velocity material beneath this. |
| 5150 | 18 | Potential localised discontinuity within rock suggested by resistivity data |
| 5290 | 23 | Significant step noted in the seismic data (Seismic Profile 16) |
| 5410 | 23 | Significant change in rock in both increased resistivity and seismic velocity suggesting the start of the hard limestone outcrop |
| 5430-6170 | 23-27A | Variability within rock layers |
| 6170-6260 | 27A | Sink hole evident in both data sets. |
| 6480 | 28 | Apparent thickening in soil suggested by resistivity and seismics (Seismic Profile 24) |
| 6870-6950 | 32 | Apparent dip in the hard rock layer suggested by seismics (Seismic Profile 30). |
| 6080-6250 | 32-33 | Minor dips in the hard rock suggested by the seismic data (Seismic Profiles 30 & 31) |
| 7370-7420 | 33 | Apparent dip in the hard rock layer suggested by seismics (Seismic Profile 31) |
| 7435 | 33 | Small localised sinkhole visible adjacent to the profile. |
| 7800-7850 | 36 | Minor dip in hard rock suggested by seismic data (Seismic Profile 33). Contrast in velocities is not strong. As such this presented with low confidence. |
| 8020-8065 | 37 | Localised dip in seismic velocity suggests a dip in the hard rock.(Seismic Profile 34) |
| 8095-8260 | 37-38 | Resistivity data shows a near surface conductive layer. Seismic data is consistent with hard rock and thin soils, suggesting this represents more weathered or saturated rock. |
| 8260 | 38 | Apparent step up in bedrock - on the edge of a profile so not presented with a high degree of certainty |
| 8580 | 38 | Significant change in resistivity values of the rock suggesting a change composition. Seismic data suggests a thickening of the overlying soil but no significant change in material velocities, suggesting a change to more weathered or saturated rocks. |

Resistivity and seismic data appears to suggest a number of significant changes in geology along the route of the pipeline. These have also been noted in above table.

### 6.1.2 2015 campaign

Geophysical investigation performed on profile PR06 indicates a lateral, dipping westward, contact between geological units with different physical characteristics, around distance of 150 meters, from the pseudo-section origin, in fact:

- the resistivity imaging prospecting shows lower resistivity values, ranging from about 20 to over 160 Ohm per meters, close to higher resistivity soils (up to 6000 Ohm x m);
- seismic refraction survey confirms lateral contact, above mentioned.

Joining both electric and seismic results, the area from distance of 60 to 150 m, seems heterogeneous, and moreover from distance of 110 to 150 meters there is a probable karst morphology.

Profile PR05, divided in two parts (PR05_A and PR05_B) due to the presence of a road, shows:

- the presence of a low resistivity and low velocity of P waves zone, centered at a distance of 360 m, whose shape suggests it is a sinkhole;
- a layer with low resistivity (about 60 Ohm*m), likely associate to underground karst, among chainage 420 and 640 meters;
- other anomalies found around chainage 90 meters and 900 meters, characterized by low seismic velocity values and very high resistivity values, suggest the presence of altered or fractured materials, related to karst phenomena.

Analysis of profile PR04, which was also divided in two sub-profiles PR04_A and PR04_B for logistic reasons, highlights:

- the presence of an higher resistivity horizon, until distance of 230 m, in contact laterally to materials characterized by lower values of resistivity, which suggests the presence of geological contact;
- from the same distance and at greater depth, the presence of a refractor with velocity of seismic wave over 2000 m/s, which suggests the presence of a material with high consistence (probably rocky);
- the presence of two anomalies, both in resistivity and in seismic wave field, at distance of 230-250 m and from 450 m to the end of profiles, which show the concomitant presence of low resistivity and low velocity of P waves, as well as the shape of anomalies, suggests karst features.

Profile PR03 highlights:

- the presence of two layers, the shallower between the ground surface and depth of about 10 meters, characterized by higher resistivity values; the second one with lower resistivity; the range resistivity values suggests the presence of soils in saturation conditions and/or characterized by a variable percentage of sandy-silty or clay fraction.
- from the distance of 50 meters towards the final part of the profile, various anomalies of high resistivity and a lower seismic velocity anomaly, which suggest the presence of altered or fractured materials.

Analysis of Profile PR02 reveals the following:

- presence of an higher resistivity horizon, which is above with a more conductive layer, at an average depth of 10 meters from the surface, in continuity with the previous profile (PR03), very close to PR02;
- an increase in speed of seismic waves downward, which suggests the presence of a material with high consistency (probably rocky);

- a contemporary decrease of the electrical resistivity, suggesting saturation conditions or variable percentage of sandy-silty fraction or clay, not excluding the presence of both conditions;

- an interruption the lateral continuity of the resistive/high velocity layer between chainages 120 and 140 meters, which, together with the shape of the electric anomaly and the trend of the seismic velocities, suggests the presence of a karst feature or an ancient riverbed buried, towards the Cassano's swamp.

Analysis of profile PR01 shows the presence of a first highly resistive layer, characterized by higher resistivity values, which thickness goes from 5 to 15 meters, placed above a second layer which resistivity is lower. Same result is given by seismic profile, in which is present a surface layer with P-wave velocity often less than 1000 m/s, whose thickness does not exceed 6 meters.

The trend of the electrical resistivity with respect to depth suggests conditions of saturation or a variable percentage of sandy-silty fraction or clay, which does not exclude the presence of both conditions.

### 6.1.1 2015 MASW profiles

Geophysical investigation by use of MASW Technique has been carried out with the aim of determining the VS30 Velocity of the shear waves in order to provide the seismic soil category.

It was performed at about Kp 0.3 along the pipeline, whose results are collected in Attachment 6.


**Figure 6-1: Layout of MASW profile.**

Soil characterization, especially from a seismic point of view and generally from a dynamic point of view, requires, the knowledge of the speed profile of Vs shear waves of soil layers on the site, down to a depth of at least 30 meters from the ground level, as required by Eurocode 8 recommendations.

The profile of Vs shear waves in the first 30 meters of depth allows to evaluate:

- the design seismic action at the foundation level of any structure
- the liquefaction risk of the soil on site
- the instability risk of slopes and/or of support structures
- the sinking of road rises, supporting works, and building foundations
- the transmission of vibrations generated by trains, vibrating machines, surface or underground explosions, vehicle traffic

Based on the speed profile of Vs shear waves in the first 30 meters of depth, it is possible to determine a speed equal to Vs30 and representing the site under examination where:

$$V_{s30} = \frac{30}{\sum_{i=1,N} \frac{h_i}{V_{Si}}}$$

where:

$h_i$ and $V_i$ denote the thickness and shear-wave velocity (at a shear strain level of $10^{-5}$ or less) of the i-th formation or layer, in a total of N, existing in the top 30 m.

The following table contains the soil categories subdivision according to NTC (as well as Eurocode 8).

**Table 6-3: Vs 30 values for main site classes according to NTC**

| Site class | $Vs_{30}$ [m/s] |
|---|---|
| A – Rock or other rock-like geological formation | > 800 |
| B – Deposits of very dense sand, gravel, or very stiff clay (Stiff Soil) | 360 – 800 |
| C – Deep deposits of dense or medium-dense sand, gravel or stiff clay (Soft Soil) | 180 – 360 |
| D – Deposits of loose-to-medium cohesionless soil (Very Soft Soil) | < 180 |
| E – A soil profile consisting of a surface alluvium layer (Alluvional) | Vs *values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with* Vs > 800 m/s |

The results of the MASW survey is:

$Vs_{30}$ = 550 m/s - Soil classification according to Eurocode 8 “B” (360 m/s < $Vs_{30}$ < 800 m/s)

## 6.2 Geotechnical investigation

### 6.2.1 Geological characterization

The below described geological model has been built on the basis of geotechnical and geophysical investigation described in Chapter 5. To complete the geological cross-section 10-11 (see Attachment 3), Piezo2, ST-BH1 and ST-BH2 stratigraphic logs were also considered.

The above reported soil investigation conducted along the onshore pipeline route allowed to define the following stratigraphic sequence, from ground surface to the investigated depth (10 m bgl):

a) red soil (local name "Terra rossa"), composed by residual sandy silt and/or clayey silt, and sometimes silty sand, with thickness ranging from few centimetres (on elevations) to around 2 metres (in depressions), discontinuously covering the units below;

b) yellowish sand, sandy silt or silty sand, sometimes clayey silt (b1), with often whitish silty sand found on the bottom, outcropping from about Kp 4.7 to the coast and found in the boreholes BH3B, BH1B_ter, BH9_bis, BH9, BH8, BH7_bis, BH7, BH6, BH5, BH4, BH3, Piezo2, ST-BH1, ST-BH2; this is mainly covered by, but sometimes covers, yellowish, soft calcarenite (b2), generally quite fractured and weathered, which outcrops along the pipeline from Kp 4.75 back to Kp 1.75 and has been found in boreholes BH3B, BH9_bis, BH7, BH5, BH4, Piezo2, ST-BH1, ST-BH2; the observations done on cores and the locations of boreholes make it likely to state that they are heteropic members within the same unit which can be correlated to "Calcarenite del Salento" formation, also known as "Calcarenite di Gravina"; from borehole BH3 backward to ST-BH2.

c) whitish, hard calcarenite, less fractured and weathered on average, outcropping from about Kp 7.7 to 7.55, from Kp 7.1 to 4.75 and encountered at very shallow levels in boreholes BH3B, BH1B_ter, BH1B_bis, BH1B, BH11_ter, BH11_bis, BH11, BH10, BH9ter, correlated to "Calcareniti di Andrano" Formation; indications of its presence in subsoil between Kp 2.7 back to 1.6, above unit b and at depths ranging from about 12 to 20 m bgl, come from geophysical investigation.

Some resistivity or P-wave velocity anomalies were found in subsoil which have been interpreted as karst cavities filled with very fractured and/or weathered material and/or in water saturation conditions. In addition, presence of little empty cavities has been found at depths ranging from 7.20 to 8.30 m bgl and in the boreholes BH1B and BH10B; these features have been interpreted like empty karst cavities. For details refer to IAL00-URS-000-Q-TRG-0001_00 (Geophysical Investigation Italy) and Attachment 4, 5 and 6 (2013 and 2015 geophysical investigation reports and profiles).

Rock quality does not seem very good, because of the widespread fracturing and weathering (particularly micro-karst) observed in the cores, even though it can be stated to be better for whitish calcarenite (on average fair rock quality) rather than for yellowish calcarenite (on average poor); for details see Attachment 08 (Borehole Logs).

A groundwater level has been found inside sand from unit b2 during the soil boring in BH7 (7.00 m bgl), BH6 (5.00 m bgl), BH5 (7.00 m bgl), BH4 (6,52 m bgl), BH3 (2.50 m bgl), Piezo3 (2.40 m bgl), ST-BH1 (2.30 m bgl) and ST-BH2 (3.96 m bgl).

### 6.2.2 Geotechnical characterization

Geotechnical characterization of pipeline was performed by means of:

- SPTs and laboratory testing on the collected samples of soil,
- RQD rate on rock cores.

SPTs were mainly performed on soils belonging to unit b1 except for BH11_ter SPT1 and BH1B (brown sandy silt/weathered and fractured calcarenite), BH5 SPT1 (very fractured and weathered yellowish calcarenite), BH11 SPT1 (brown sandy silt).

Nspt values range from 7 to above 50 in tests performed in unit b1, thus they can be classified as *loose* to *very dense soils* (Terzaghi-Peck, 1948), while their friction angle goes from 25,5 to 34,9 ° (De Mello, 1971); for details see the following Table 6-4 and Attachment 12.

**Table 6-4: summary of SPTs performed in unit a) and b1) and of geotechnical parameters derived from correlations with Nspt**

| BOREHOLE | SPT | DEPTH, m bgl | UNIT | $N_{SPT}$ | $(N_1)_{60}$ | ϕ, ° (De Mello 1971) | $D_R$, % (Bazaraa 1967) | Young Modulus E, Mpa (Jambu) | Shear Modulus G, Mpa (Ohsaki & Iwasaki) | Oedometric Modulus $E_{ed}$, Mpa |
|---|---|---|---|---|---|---|---|---|---|---|
| BH11 | SPT1 | 0.50÷0.95 | a | 4 | 3.0 | 23.0 | 19.4 | 15.7 | 28.0 | 2.2 |
| BH11_TER | SPT1 | 0.50÷0.95 | a/c | 65 | 52.0 | 33.9 | 135.9 | 7.6 | 233.3 | 36.2 |
| BH1B | SPT1 | 0.50÷0.95 | a/c | 51 | 41.0 | 33.0 | 120.4 | 7.6 | 194.1 | 28.4 |
| BH8 | SPT1 | 0.50÷0.95 | a/b1 | 51 | 41.0 | 33.0 | 123.3 | 6.9 | 194.1 | 28.4 |
| BH3 | SPT1 | 3.00÷3.45 | b1 | 36 | 24.0 | 32.2 | 74.8 | 28.0 | 169.7 | 23.8 |
| BH3 | SPT2 | 6.20÷6.65 | b1 | 10 | 10.0 | 27.4 | 36.4 | 18.1 | 55.0 | 6.9 |
| BH3 | SPT3 | 9.50÷9.95 | b1 | 18 | 18.0 | 29.1 | 42.5 | 31.3 | 97.5 | 11.5 |
| BH3B | SPT1 | 1.50÷1.95 | b1 | 78 | 68.0 | 34.9 | 129.7 | 14.7 | 337.4 | 47.5 |
| BH4 | SPT1 | 4.00÷4.45 | b1 | 12 | 12.0 | 28.1 | 43.0 | 18.3 | 63.5 | 8.0 |
| BH4 | SPT2 | 6.00÷6.45 | b1 | 38 | 38.0 | 31.1 | 59.7 | 36.0 | 137.0 | 18.2 |
| BH5 | SPT2 | 7.00÷7.45 | b1 | 73 | 44.0 | 33.0 | 72.1 | 28.8 | 205.5 | 30.6 |
| BH6 | SPT1 | 1.00÷1.45 | b1 | 91 | 45.0 | 33.4 | 115.6 | 12.0 | 209.2 | 31.3 |
| BH6 | SPT2 | 2.80÷3.25 | b1 | 70 | 40.0 | 32.8 | 82.7 | 20.0 | 191.0 | 27.8 |
| BH6 | SPT3 | 7.00÷7.45 | b1 | 24 | 20.0 | 29.9 | 48.0 | 28.8 | 110.7 | 13.6 |
| BH7BIS | SPT1 | 0.50÷0.95 | b1 | 7 | 6.0 | 25.5 | 45.7 | 8.5 | 42.9 | 3.9 |
| BH5 | SPT1 | 1.00÷1.45 | b2 | 51 | 28.0 | 31.6 | 91.2 | 18.5 | 146.0 | 19.5 |

Both undisturbed and disturbed samples were collected in the sandy-silty member of unit b1 except for samples BH1B_ter C1 and BH11 C1, which were collected in unit a); they have undergone to laboratory testing in order to perform the following determinations: Natural unit weight, Dry unit weight, Water content, Specific gravity, Porosity, void ratio, Degree of saturation, Particle size distribution, Atterberg's plasticity tests, Hydraulic conductivity by Oedometer; for details see the following Table 6-5 and Attachment 11.

**Table 6-5: Summary of laboratory test performed in b1)**

| Sample code | Depth of collection (m bgl) | γn (KN/m$^3$) | γd (KN/m$^3$) | W (%) | G (-) | n (%) | E (-) | S (%) | k (m/sec) |
|---|---|---|---|---|---|---|---|---|---|
| BH 1B ter-C2 | 4.00-4.50 | - | - | 22.85 | 2.64 | - | - | - | |
| BH3-C1 | 3.00-3.45 | - | - | 18.84 | - | - | - | - | |
| BH3-C2 | 6.20-6.65 | - | - | 26.36 | - | - | - | - | |
| BH3-C3 | 9.50-9.95 | - | - | 14.43 | - | - | - | - | |
| BH4-C1 | 3.50-4.00 | 17.34 | 14.69 | 18.1 | 2.71 | 45.82 | 0.85 | 58.01 | 9.11E-06 |
| BH4-C2 | 4.00-4.54 | - | - | 7.53 | - | - | - | - | |
| BH4-C3 | 6.50-6.95 | - | - | 17.7 | - | - | - | - | |
| BH 5-C1 | 2.80-3.40 | 14.25 | 10.96 | 30.08 | 2.65 | 58.60 | 1.42 | 56.23 | 5.13E-06 |
| BH 6-C1 | 6.00-6.40 | 17.19 | 13.24 | 29.82 | 2.65 | 49.96 | 1 | 79.02 | 5.97E-07 |
| BH 6-C2 | 7.00-7.45 | - | - | 22.89 | 2.67 | - | - | - | |
| BH 7-C1 | 3.00-3.50 | 17.01 | 13.58 | 25.25 | 2.61 | 48.07 | 0.93 | 71.34 | 9.96E-07 |
| BH 7-C2 | 7.00-7.45 | - | - | 30.44 | 2.65 | - | - | - | |
| BH 7 bis-C1 | 3.00-3.30 | - | - | 13.36 | 2.71 | - | - | - | |
| BH 7 bis-C2 | 3.80-4.40 | 14.30 | 11.78 | 21.42 | 2.66 | 55.76 | 1.26 | 45.26 | 1.72E-06 |
| BH 8-C1 | 4.00-4.50 | 18.03 | 15.99 | 12.73 | 2.65 | 39.64 | 0.66 | 51.37 | 1.93E-06 |
| BH 8-C2 | 5.00-5.40 | - | - | 21.52 | 2.70 | - | - | - | |
| BH 9-C1 | 6.00-6.40 | - | - | 15.66 | 2.69 | - | - | - | |
| BH 9 bis-C1 | 7.00-7.50 | 18.60 | 14.6 | 27.37 | 2.62 | 44.32 | 0.80 | 90.17 | 1.03E-05 |

The prevailing particle size classes are sand, secondly gravel, then silt and finally clay, their mean distribution provides a *silty sand with gravel*, while Atterberg test results indicate a non plastic behaviour which consistently reflects the prevailing of coarse with respect to fine classes in size distribution.

Natural unit weight ranges from 14,25 to 18,60 kNm$^{-3}$, with a mean value of 16,67 kNm$^{-3}$, certainly correlated with the medium to high values of porosity and void ratio. Specific gravity has a mean value of 26,6 and varies from 26,1 to 27,1, being consistent with the prevailing mineralogical composition of grains.

Water content ranges from 7,53 to 30,44%, with a mean value of 20,91%, while saturation degree varies between 45,26 and 90,17%, with a mean value of 48.88. This is consistent with the overall state of subsoil, particularly in the first half of pipeline route (Kp 0 to 3.2) in which a groundwater level was found.

Hydraulic conductivity values within the low hydraulic conductivity class and are typical of fine sand, sandy silt or silty sand.

Geotechnical characterization of the units b2) (yellowish calcarenite belonging to the formation of Calcarenite Gravina) and c) (whitish calcarenite, belonging to the formation of Calcarenite Andrano) was made on the basis of the Rock Quality Designation (RQD) index, which is the measure of the degree of jointing or fractures in a rock mass, measured as a percentage of the drill core in lengths of 10 cm or more.

Yellowish calcarenite was encountered in 10 boreholes (BH3B, BH9_bis, BH9, BH7, BH5, BH4, BH3, Piezo2, ST-BH1, ST-BH2) along pipeline route, at depths ranging from 0,00 to 9.00 m bgl. It is characterized from 0 to 46% (from very poor to fair) RQD values, distributed per RQD class as follows:

- 0-20 % class: 87.%
- 20-40 % class: 6.3%
- 40-60 % class: 6.3%
- 60-80 % class: 0%
- 80-100 % class: 0%

**Table 6-6: distribution of RQD rates vs. Depth for unit b2)**

| DEPTH<br>RQD class | 0-3 m | 3-6 m | 6-9 m | 9-10 m |
|---|---|---|---|---|
| very poor | 8 | 4 | 2 | 0 |
| Poor | 1 | 0 | 0 | 0 |
| Fair | 1 | 0 | 0 | 0 |
| Good | 0 | 0 | 0 | 0 |
| Excellent | 0 | 0 | 0 | 0 |

Whitish calcarenite was instead found in 9 boreholes (BH3B, BH1B_ter, BH1B_bis, BH1B, BH11_ter, BH11_bis, BH11, BH10, BH9_ter), with top ranging from 0 to 3,00 m bgl. They are characterized by RQD values ranging from 0 to 70% (from very poor to good), distributed as follows

- 0-20% class: 41.2%
- 20-40% class: 26.5,2%
- 40-60% class: 23.5%
- 60-80% class: 8.8%
- 80-100% class: 0.0%

**Table 6-7: distribution of RQD rates vs. Depth for unit c)**

| DEPTH<br>RQD class | 0-3 m | 3-6 m | 6-9 m | 9-10 m |
|---|---|---|---|---|
| very poor | 3 | 3 | 3 | 5 |
| poor | 4 | 1 | 4 | 0 |
| fair | 0 | 2 | 2 | 4 |
| good | 0 | 3 | 0 | 0 |
| excellent | 0 | 0 | 0 | 0 |

As it can be noticed, whitish calcarenite looks to have slightly better engineering properties than the yellowish one.

According to classes of workability of DIN 18300 "Earthwork", the classes that should be considered are:

- n° 6 (*rock which is easy to excavate and similar soils; more than 30 % stones with each 0,01 to 0,1 m*$^3$*; solid clay and silt*);
- n° 7 *(rock which is hard to excavate)*.

Based on geotechnical boreholes performed, some of which are very far from each others, and the geological interpretation represented in Attachment 3, class n° 6 is suggested from chainage 0 m to 3790 m (down to the investigated depth, 10 m bgl) and for yellowish calcarenite; meanwhile class n° 7 is suggested from chainage 3790 m to 8235 m (launch shaft for Microtunnel, down to the investigated depth, 10 m bgl) (Ref. Attachment 3).

However it is not possible to exclude that different soil/rock conditions may be encountered along the whole pipeline trench.

## 6.3 Disaggregated and characteristic values of geotechnical parameters

The results of the on-site and laboratory tests have been processed in order to get statistical indicators like mean, median, min, max, standard deviation, coefficient of variation, etc. per each geotechnical unit.

As soils of unit a) were found in boreholes with poor thickness and sometimes directly laying on yellowish or whitish calcarenite:

- SPT values are affected by the underlaying calcarenites except one (BH11 SPT1),
- it was not possible to take undisturbed or semidisturbed samples of such an amount to perform determination of $\gamma_n$ and $\gamma_d$,

only the SPT value above mentioned and the values of $\gamma_n$ and $\gamma_d$ determined from laboratory tests in the PRT, given the similarity of the materials, were used along with the other parameters.

It has also to be noticed that parameters of unit b1) derived from Nspt show a remarkable dispersion.

**Table 6-8: statistical indicators for parameters derived from correlation with Nspt per unit**

| Unit | | $N_{spt}$ | $(N_1)_{60}$ | $\phi$ | $D_R$ | $E_d$ | G | $E_{ed}$ |
|---|---|---|---|---|---|---|---|---|
| a | count | 4.00 | 3.00 | 23.03 | 19.38 | 15.72 | 28.04 | 2.23 |
| | Mean | 4.00 | 3.00 | 23.03 | 19.38 | 15.72 | 28.04 | 2.23 |
| | Min | 4.00 | 3.00 | 23.03 | 19.38 | 15.72 | 28.04 | 2.23 |
| | Max | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| | St. Dev. | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| | COV | 1 | 1 | 1 | 1 | 1 | 1 | 1 |
| b1 | count | 42.33 | 30.50 | 30.87 | 72.80 | 20.96 | 151.14 | 20.96 |
| | Mean | 7.00 | 6.00 | 25.50 | 36.44 | 6.93 | 42.90 | 3.90 |
| | Min | 91.00 | 68.00 | 34.92 | 129.75 | 36.01 | 337.44 | 47.49 |
| | Max | 29.58 | 18.41 | 2.86 | 33.55 | 9.52 | 84.30 | 12.87 |
| | St. Dev. | 0.699 | 0.604 | 0.092 | 0.461 | 0.454 | 0.558 | 0.614 |
| | COV | 12 | 12 | 12 | 12 | 12 | 12 | 12 |

**Table 6-9: statistical indicators for parameters derived from laboratory tests for unit b1**

| | $\gamma_n$ | $\gamma_d$ | W | G | n | e | S | k (m/sec) |
|---|---|---|---|---|---|---|---|---|
| count | 7 | 7 | 18 | 13 | 7 | 7 | 7 | 7 |
| average | 16.67 | 13.55 | 20.91 | 2.66 | 48.88 | 0.99 | 64.49 | 4.25E-06 |
| min | 14.25 | 10.96 | 7.53 | 2.61 | 39.64 | 0.66 | 45.26 | 5.97E-07 |
| max | 18.60 | 15.99 | 30.44 | 2.71 | 58.60 | 1.42 | 90.17 | 1.03E-05 |
| median | 17.19 | 13.58 | 21.47 | 2.65 | 48.07 | 0.93 | 58.01 | 1.93E-06 |
| st. dev. | 1.73 | 1.75 | 6.63 | 0.03 | 6.57 | 0.27 | 16.17 | 4.01E-06 |
| COV | 0.10 | 0.13 | 0.32 | 0.01 | 0.13 | 0.27 | 0.25 | 9.44E-01 |

Further on, according to “Eurocode 7 EN 1997-1 Geotecnical design” and to Italian regulation (mainly “Norme Tecniche sulle Costruzioni”, 2008), from these “disaggregated” values of geotechnical parameters have been derived “characteristic” values by means of statistical approach.

As EC7 states, *“...The characteristic value of a parameter of a soil or rock should be chosen on the basis of a cautionary assessment of the value which affects the occurrence of the limit state...”*; two are the foreseen approaches:

- when the limit state is controlled by the mean value of a parameter of the soil (eg. when it involves large volumes of soil and a redistribution of loads can occur) the characteristic value should be chosen as a precautionary estimate of the mean value;

- when instead the limit state interests small volumes of soil and / or not many results of tests are available for them and / or the dispersion of the values is high, then it is more correct to perform a precautionary estimate of the local lower value.

Given the planned engineering works and constructions, the geological local setting in which they will be made, the amount and the quality of data, the second approach was chosen.

Having a relatively small number of tests, the statistic approach was carried out using the following equation as suggested by H.R. Schneider, P. Fitze – (2011):

$$x_k = x_{mean} - 1.645\frac{\sigma}{\sqrt{n}})$$

Where:

$x_{mean}$ is the arithmetic mean,

$\sigma$ is the standard deviation,

$n$ is the number of tests

Using the above equation, the characteristic values reported in following Table 6-12 and Table 6-13 were obtained.

**Table 6-10: characteristic values for parameters derived from correlation with SPT per unit a and b1**

| Unit | $N_{spt,k}$ | $(N_1)_{60,k}$ | $\phi_k$ | $D_{R,k}$ | $Ed_{,k}$ | $G_k$ | $E_{ed,k}$ |
|---|---|---|---|---|---|---|---|
| a | 4,00 | 3,00 | 23,03 | 19,38 | 15,72 | 28,04 | 2,23 |
| b1 | 28.29 | 21.76 | 29.52 | 56.87 | 16.44 | 111.11 | 14.85 |

**Table 6-11: characteristic values for parameters derived from laboratory tests for unit b1**

| Natural unit weight, $\gamma n$ (KN/m$^3$) | Dry unit weight, gd (KN/m$^3$) | Water content, W (%) | Specific gravity, G (-) | Porosity, n (%) | void ratio, e (-) | Degree of saturation, S (%) | $k_k$ (m/sec) |
|---|---|---|---|---|---|---|---|
| 15.60 | 12.46 | 18.34 | 2.65 | 44.80 | 0.82 | 54.43 | 1.7585E-06 |

## 6.4 Design seismic actions

The new Italian building code NTC (Norme Tecniche NTC 2008) covers several topics, including the design of new civil and industrial constructions, bridges and geotechnical structures and the modification of existing structures.

First, it introduces a reference period $V_R$ for seismic actions, which is given by the product of the nominal life of a construction $V_N$ and its coefficient of use $C_U$. $V_N$ is the number of years during which a structure, if subjected to regular maintenance, should be used for the purpose for which it was designed. It is suggested that $V_N$ = 10 years for temporary structures, $V_N \geq 50$ years for ordinary buildings and structures, and $V_N \geq 100$ years for large or strategic constructions.

The coefficient of use is directly linked to the class of use of the construction, from Class I (rare presence of people, construction for agriculture, $C_U = 0.7$) to Class II (normal presence of people, $C_U = 1.0$) up to Class IV (important public and strategic buildings also used for civil protection, $C_U = 2.0$).

Two damage limit states (SLO = Operability limit state, SLD = Limit state of prompt use or Damage (SLD) and two ultimate limit states (SLU = Limit state for the safeguard of human life or Ultimate state, SLC = Limit state for collapse prevention (SLC) are established in the code.

According to the code, the probability of exceedance of the seismic action during the reference period varies with the limit state.

**Table 6-12: variation of the probability of exceedance of the seismic motion for different limit states**

| Limit state | | Probability $P$ of exceedance in the reference period $V_R$ |
|---|---|---|
| Serviceability limit state | SLO | 81% |
| | SLD | 63% |
| Ultimate limit state | SLU | 10% |
| | SLC | 5% |

This way of defining the earthquake returning period is associated with a system that has recently become available in Italy, which allows visualization and querying of probabilistic seismic hazard maps of the national territory using several shaking parameters on a regular grid with a 0.05° spacing (Meletti and Montaldo, 2007).

In summary, there is now a tool in Italy, incorporated into the NTC that allows determination of the PGA and the design spectrum at each location in the territory for earthquakes with different returning periods.

### 6.4.1 Subsoil categories

The Geotechnical Earthquake Engineering community is well aware that local soil conditions can greatly modify seismic motion characteristics from those on outcropping bedrock.

In the NTC, site effects are introduced through the determination of ground type, which influences the soil factor and the shape of the design response spectrum.

Especially, the equivalent shear wave velocity Vs30 is introduced, which has been strongly recommended, and an equivalent NSPT30 and an equivalent Cu30 are defined.

A clearer definition of the soil depth for which these equivalent parameters may be evaluated is given according to the construction type. The depth should be computed from the embedment depth for shallow foundations; from the pile head for deep foundations; from the wall head for retaining walls for natural soils; and from the depth of the foundation for retaining walls for earthworks.

As for the ground type, it is specified that a deposit can be classified into one of the five conventional categories (from class A to class E) only if a regular increase in its mechanical properties with depth is observed. If not, the site should be classified as S2 and special studies for definition of the seismic action are required.

**Table 6-13: Vs 30 values for main site classes according to NTC**

| Site class | $Vs_{30}$ [m/s] |
|---|---|
| A – Rock or other rock-like geological formation | > 800 |
| B – Deposits of very dense sand, gravel, or very stiff clay (Stiff Soil) | 360 – 800 |
| C – Deep deposits of dense or medium-dense sand, gravel or stiff clay (Soft Soil) | 180 – 360 |
| D – Deposits of loose-to-medium cohesionless soil (Very Soft Soil) | < 180 |
| E – A soil profile consisting of a surface alluvium layer (Alluvional) | Vs *values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with* Vs > 800 m/s |

In 2015 one MASW profile has been performed (MASW 3), whose results are collected in Attachment 6 as follows:

Profile 1 - *MASW 3*

$Vs_{,30}$ = 550 m/s - Soil classification according to Eurocode 8 "B" (360 m/s < $Vs_{,30}$ < 800 m/s)

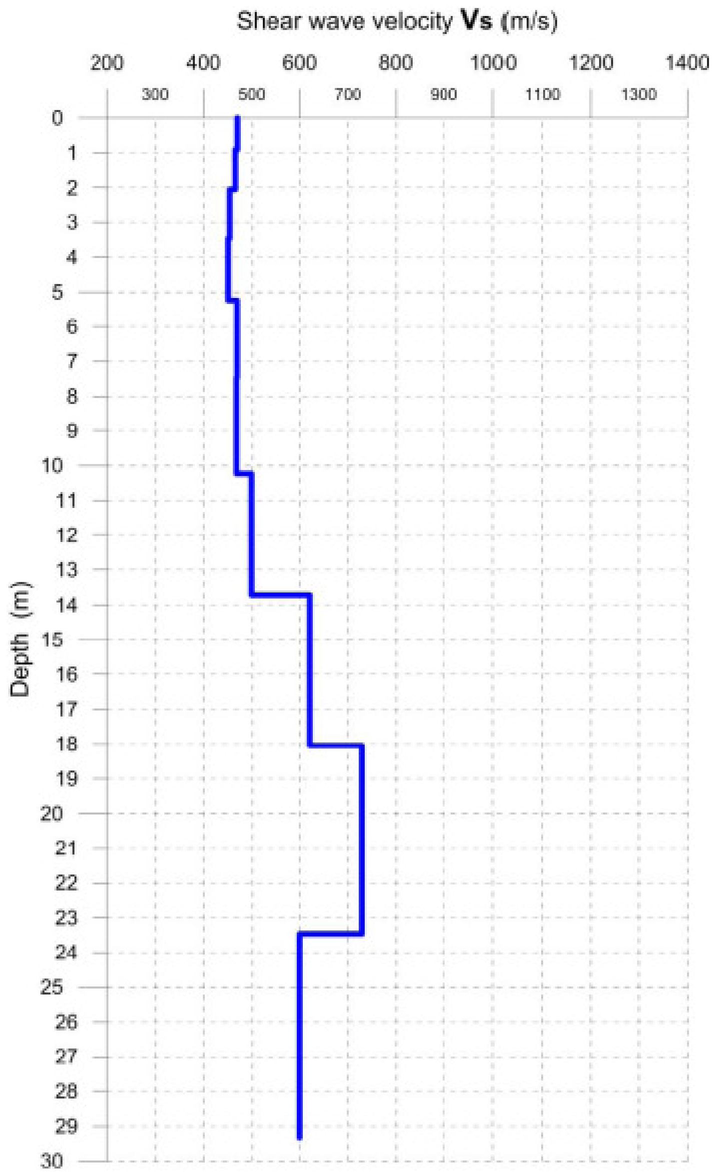



**Figure 6-2: MASW 3 profile**

**The final seismic characterization here suggested is to use category B for the site Pipeline Onshore Route (as well as for PRT area).**

### 6.4.2 Hazard identification site

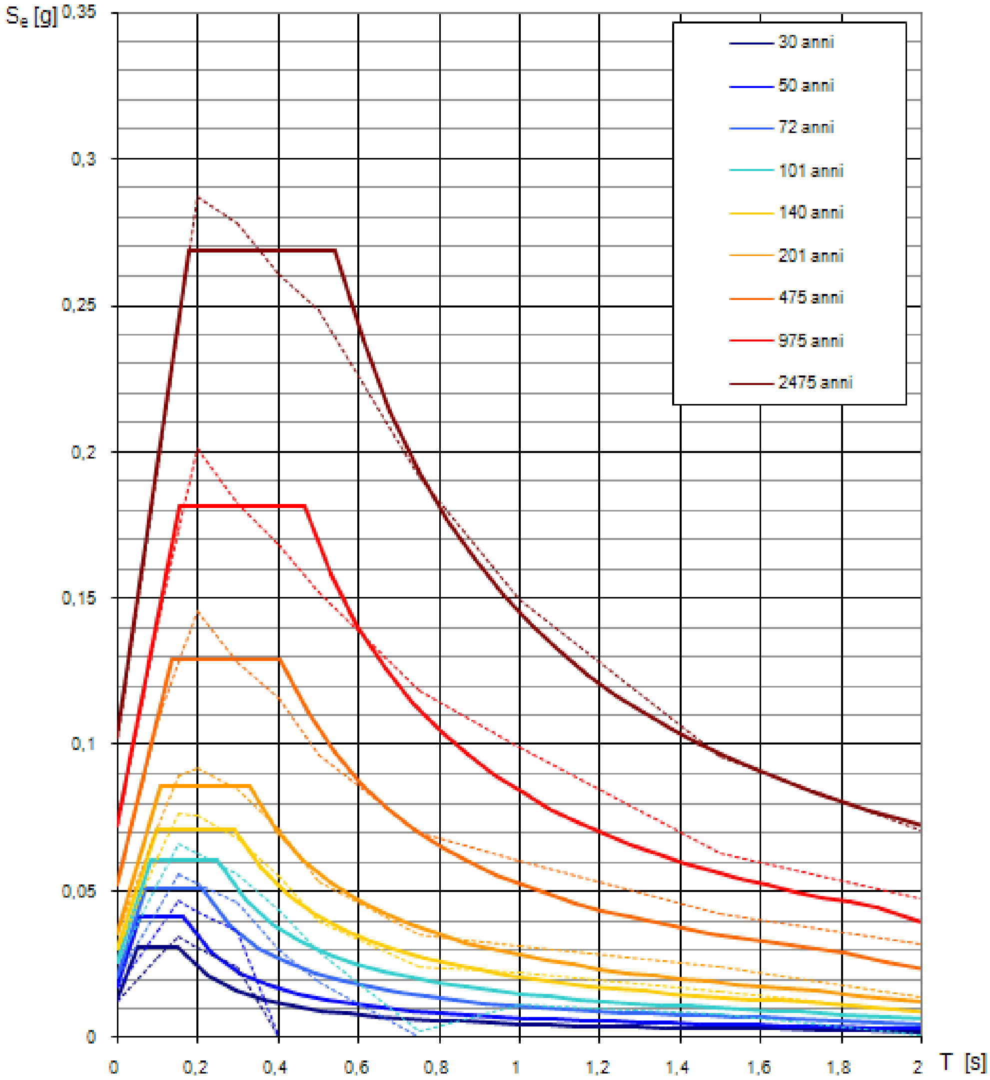
In compliance with NTC, spectral shapes are defined, for each of the probability of overshooting in reference period $V_R$, from the values of the following reference parameters:

- $a_g$: maximum horizontal acceleration at the site;
- $F_o$: maximum value of the amplification factor of the spectrum in horizontal acceleration;
- $T_c$: beginning period of the stroke at a constant speed of the spectrum in horizontal acceleration.

The municipality of Melendugno is located in seismic zone 4 (according to DGR no 153/2004, we consider zone 3 for strategic structures), with seismic parameters for reference return periods Tr, reported in the following table:

**Table 6-14: parameter values ag, Fo, Tc for the reference return periods**

| $T_R$ [anni] | $a_g$ [g] | $F_o$ [-] | $T_C^*$ [s] |
|---|---|---|---|
| 30 | 0,013 | 2,458 | 0,152 |
| 50 | 0,017 | 2,436 | 0,163 |
| 72 | 0,021 | 2,441 | 0,213 |
| 101 | 0,026 | 2,362 | 0,249 |
| 140 | 0,030 | 2,355 | 0,291 |
| 201 | 0,035 | 2,420 | 0,328 |
| 475 | 0,052 | 2,462 | 0,406 |
| 975 | 0,072 | 2,505 | 0,464 |
| 2475 | 0,102 | 2,627 | 0,540 |




NOTA:
Con linea continua si rappresentano gli spettri di Normativa, con linea tratteggiata gli spettri del progetto S1-INGV da cui sono derivati.

**Figure 6-3: spectral shapes for the reference return periods (NCT - continuos / Project S1-INGV calculated – hatched)**

### 6.4.3 Choice of strategy design

In compliance with NTC, the strategy design in this case considers a nominal life of the building PRT equivalent to 100 years and a use coefficient $C_U$ of 2.

Below the input design values:

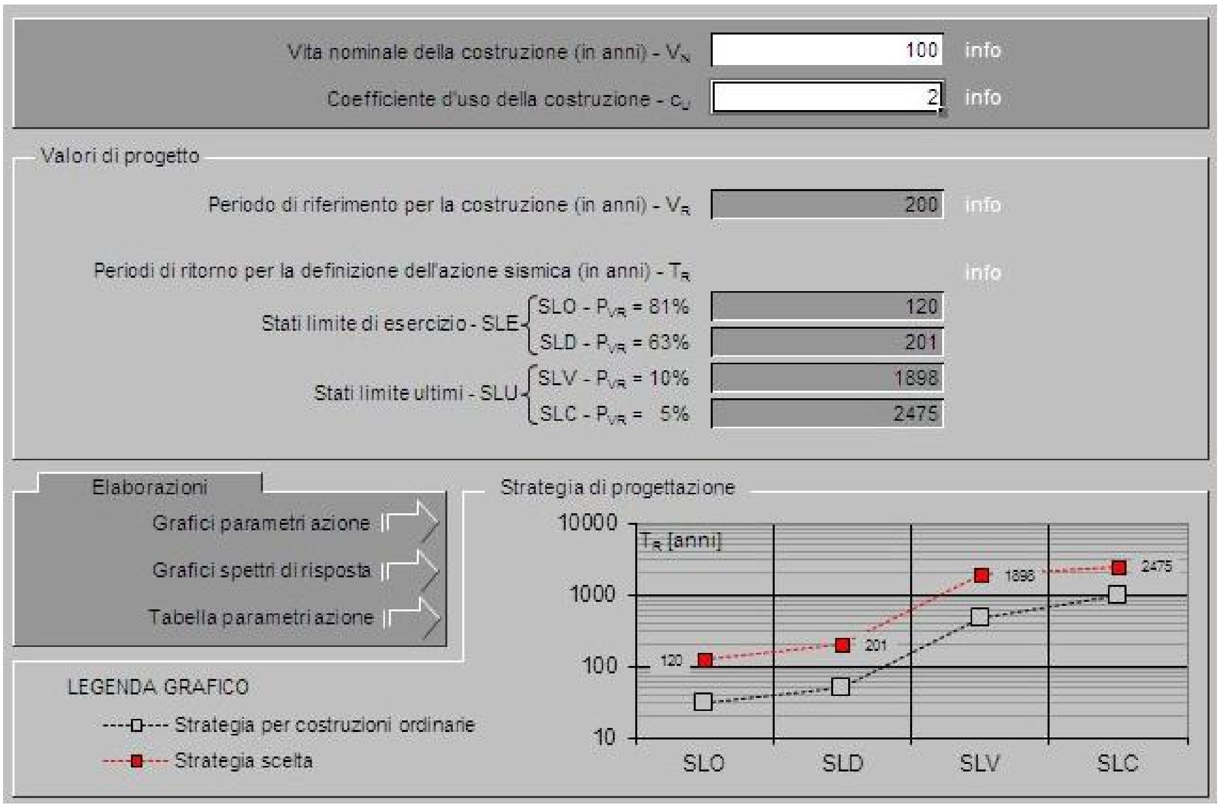



**Figure 6-4: input design strategy values (spettri NTC. Ver. 1.0.3 – Supreme Council of Public Works)**

In the Table 6-15 are represented parameters ag, Po, Tc for the four limit states, considering a nominal life of the building PRT equivalent to 100 years and a use coefficient of 2.

**Table 6-15: parameters ag, Po, Tc for the four limit states, considering a nominal life of the building PRT equivalent to 100 years and a use coefficient of 2.**

| SLATO LIMITE | $T_R$ [anni] | $a_g$ [g] | $F_o$ [-] | $T_C^*$ [s] |
|---|---|---|---|---|
| SLO | 120 | 0,028 | 2,358 | 0,271 |
| SLD | 201 | 0,035 | 2,420 | 0,328 |
| SLV | 1898 | 0,093 | 2,592 | 0,517 |
| SLC | 2475 | 0,102 | 2,627 | 0,540 |

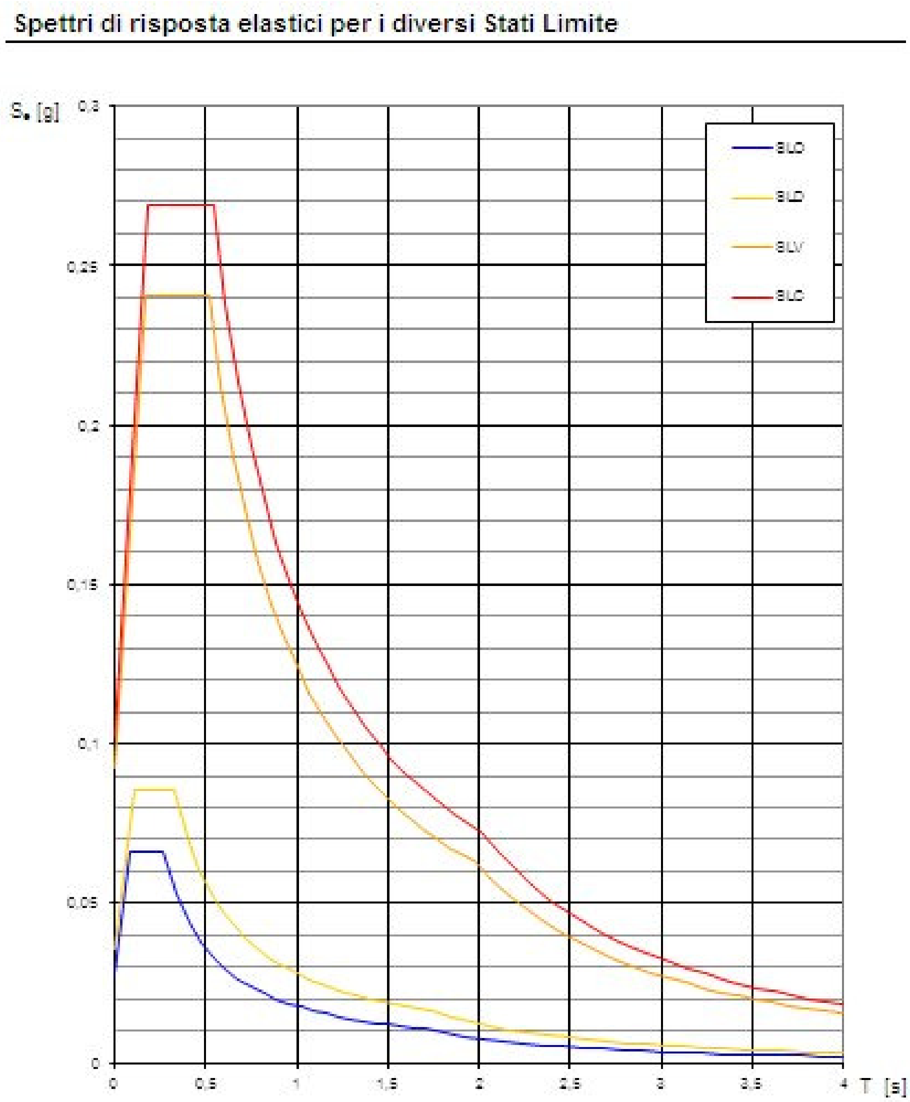



**Figure 6-5: spectral shapes for different limit states**

### 6.4.4 Determination of the design seismic action

The design seismic action is based on the identification of categories of subsoil and on the topography of the site.

**We consider subsoil category = B and Topographic category = T1 (flat surface).**

Below a figure with the input parameters considered:

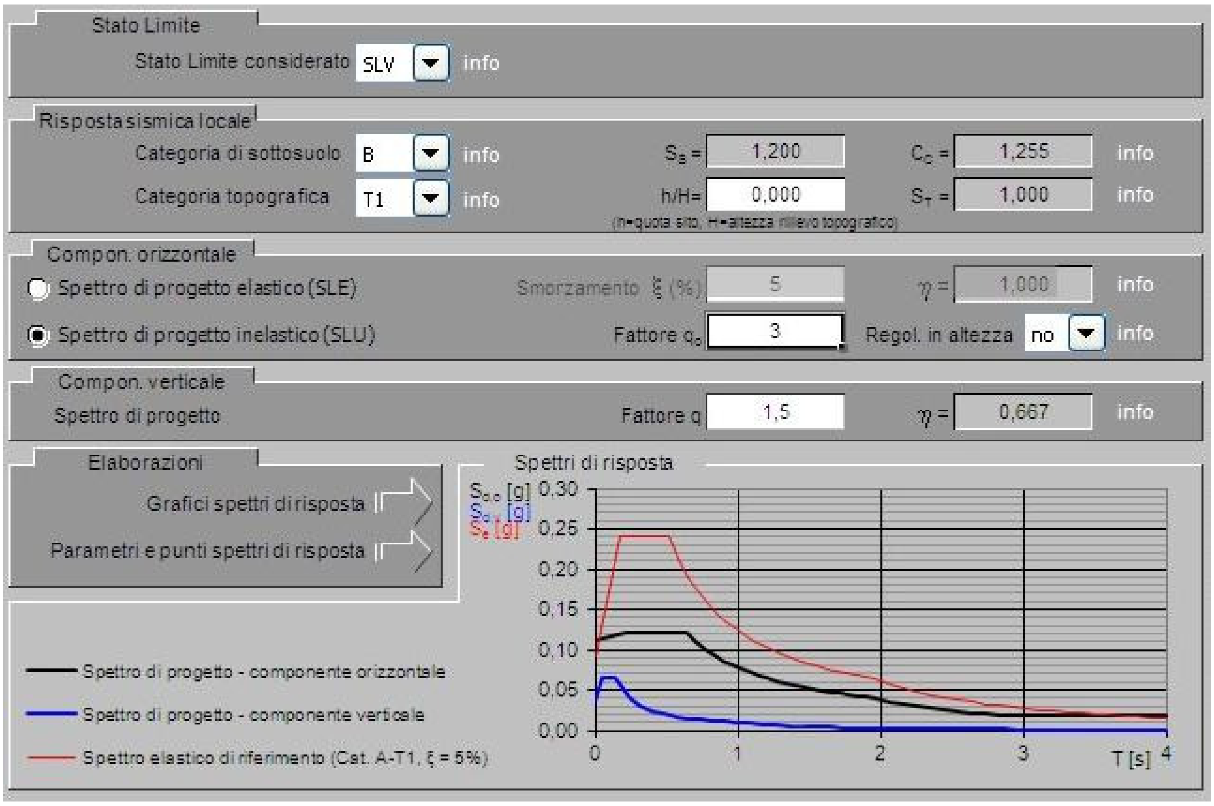



**Figure 6-6: input values for determination of the design seismic action (spettri NTC. Ver. 1.0.3 – Supreme Council of Public Works)**

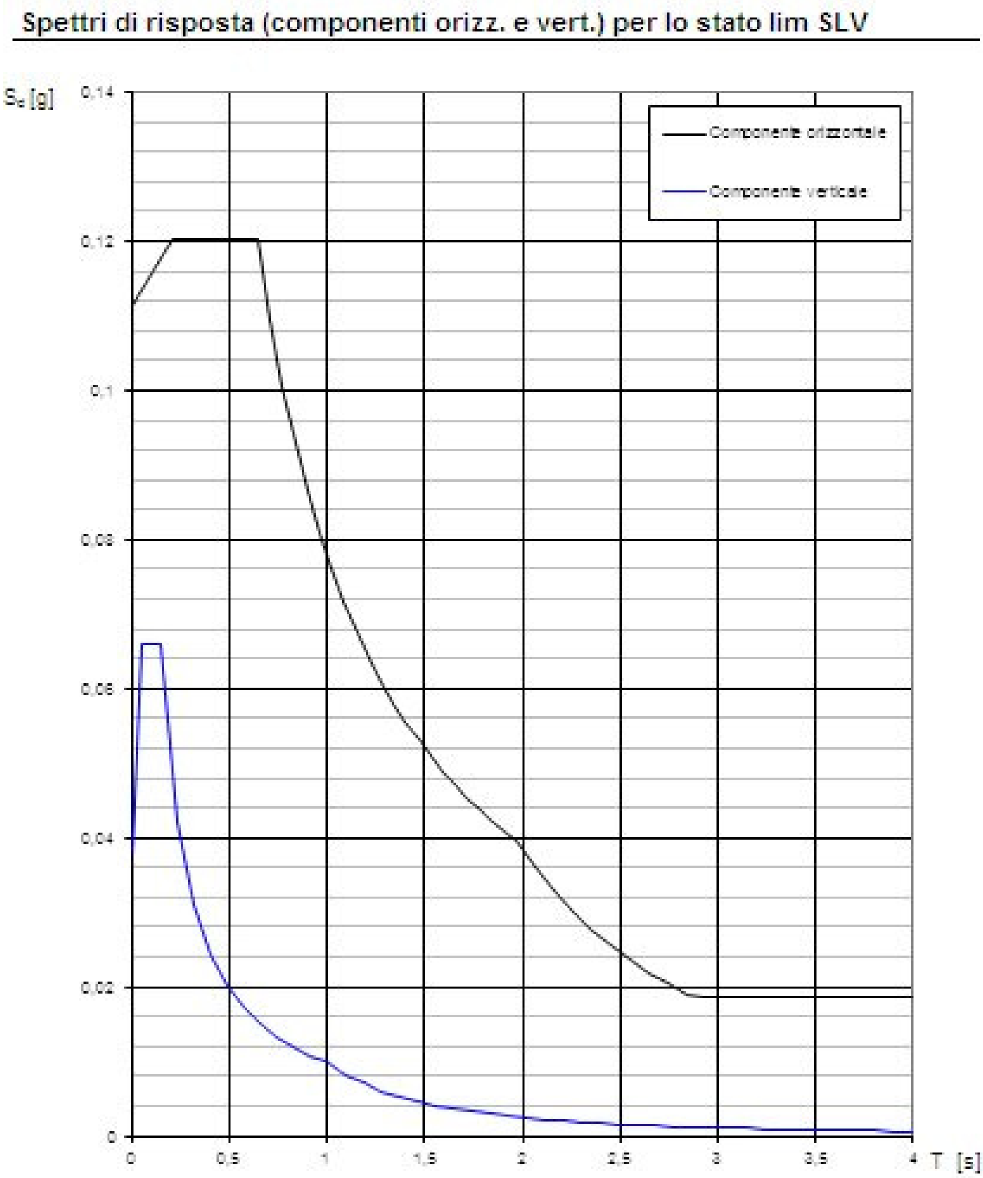



**Figure 6-7: response spectra of horizontal and vertical components for the SLV**

Parametri indipendenti

| STATO LIMITE | SLV |
|---|---|
| $a_g$ | 0,093 g |
| $F_o$ | 2,592 |
| $T_C^*$ | 0,517 s |
| $S_S$ | 1,200 |
| $C_C$ | 1,255 |
| $S_T$ | 1,000 |
| q | 2,400 |

Parametri dipendenti

| | |
|---|---|
| S | 1,200 |
| η | 0,417 |
| $T_B$ | 0,216 s |
| $T_C$ | 0,649 s |
| $T_D$ | 1,971 s |

Espressioni dei parametri dipendenti

$S = S_S \cdot S_T$ (NTC-08 Eq. 3.2.5)

$\eta = \sqrt{10/(5+\xi)} \geq 0,55;\ \eta = 1/q$ (NTC-08 Eq. 3.2.6; §. 3.2.3.5)

$T_B = T_C/3$ (NTC-07 Eq. 3.2.8)

$T_C = C_C \cdot T_C^*$ (NTC-07 Eq. 3.2.7)

$T_D = 4,0 \cdot a_g/g + 1,6$ (NTC-07 Eq. 3.2.9)

Espressioni dello spettro di risposta (NTC-08 Eq. 3.2.4)

$$0 \leq T < T_B \quad S_e(T) = a_g \cdot S \cdot \eta \cdot F_o \cdot \left[\frac{T}{T_B} + \frac{1}{\eta \cdot F_o}\left(1 - \frac{T}{T_B}\right)\right]$$

$$T_B \leq T < T_C \quad S_e(T) = a_g \cdot S \cdot \eta \cdot F_o$$

$$T_C \leq T < T_D \quad S_e(T) = a_g \cdot S \cdot \eta \cdot F_o \cdot \left(\frac{T_C}{T}\right)$$

$$T_D \leq T \quad S_e(T) = a_g \cdot S \cdot \eta \cdot F_o \cdot \left(\frac{T_C T_D}{T^2}\right)$$

Lo spettro di progetto $S_d(T)$ per le verifiche agli Stati Limite Ultimi è ottenuto dalle espressioni dello spettro elastico $S_e(T)$ sostituendo η con 1/q, dove q è il fattore di struttura. (NTC-08 § 3.2.3.5)

Punti dello spettro di risposta

| | T [s] | Se [g] |
|---|---|---|
| | 0,000 | 0,111 |
| $T_B$ ← | 0,216 | 0,120 |
| $T_C$ ← | 0,649 | 0,120 |
| | 0,712 | 0,110 |
| | 0,775 | 0,101 |
| | 0,838 | 0,093 |
| | 0,901 | 0,087 |
| | 0,964 | 0,081 |
| | 1,027 | 0,076 |
| | 1,090 | 0,072 |
| | 1,153 | 0,068 |
| | 1,216 | 0,064 |
| | 1,279 | 0,061 |
| | 1,341 | 0,058 |
| | 1,404 | 0,056 |
| | 1,467 | 0,053 |
| | 1,530 | 0,051 |
| | 1,593 | 0,049 |
| | 1,656 | 0,047 |
| | 1,719 | 0,045 |
| | 1,782 | 0,044 |
| | 1,845 | 0,042 |
| | 1,908 | 0,041 |
| $T_D$ ← | 1,971 | 0,040 |
| | 2,068 | 0,036 |
| | 2,164 | 0,033 |
| | 2,261 | 0,030 |
| | 2,358 | 0,028 |
| | 2,454 | 0,026 |
| | 2,551 | 0,024 |
| | 2,647 | 0,022 |
| | 2,744 | 0,020 |
| | 2,841 | 0,019 |
| | 2,937 | 0,019 |
| | 3,034 | 0,019 |
| | 3,130 | 0,019 |
| | 3,227 | 0,019 |
| | 3,324 | 0,019 |
| | 3,420 | 0,019 |
| | 3,517 | 0,019 |
| | 3,614 | 0,019 |
| | 3,710 | 0,019 |
| | 3,807 | 0,019 |
| | 3,903 | 0,019 |
| | 4,000 | 0,019 |

**Figure 6-8: parameters and points of the spectrum of horizontal component for the SLV**

Parametri indipendenti

| STATO LIMITE | SLV |
|---|---|
| $a_{gv}$ | 0,038 g |
| $S_S$ | 1,000 |
| $S_T$ | 1,000 |
| q | 1,500 |
| $T_B$ | 0,050 s |
| $T_C$ | 0,150 s |
| $T_D$ | 1,000 s |

Parametri dipendenti

| | |
|---|---|
| $F_v$ | 1,066 |
| S | 1,000 |
| η | 0,667 |

Espressioni dei parametri dipendenti

$$S = S_S \cdot S_T$$ (NTC-08 Eq. 3.2.5)

$$\eta = 1/q$$ (NTC-08 §. 3.2.3.5)

$$F_v = 1{,}35 \cdot F_o \cdot \left(\frac{a_g}{g}\right)^{0,5}$$ (NTC-08 Eq. 3.2.11)

Espressioni dello spettro di risposta (NTC-08 Eq. 3.2.10)

$$0 \le T < T_B \quad S_e(T) = a_g \cdot S \cdot \eta \cdot F_v \cdot \left[\frac{T}{T_B} + \frac{1}{\eta \cdot F_o}\left(1 - \frac{T}{T_B}\right)\right]$$

$$T_B \le T < T_C \quad S_e(T) = a_g \cdot S \cdot \eta \cdot F_v$$

$$T_C \le T < T_D \quad S_e(T) = a_g \cdot S \cdot \eta \cdot F_v \cdot \left(\frac{T_C}{T}\right)$$

$$T_D \le T \quad S_e(T) = a_g \cdot S \cdot \eta \cdot F_v \cdot \left(\frac{T_C T_D}{T^2}\right)$$

Punti dello spettro di risposta

| | T [s] | Se [g] |
|---|---|---|
| | 0,000 | 0,038 |
| $T_B$ | 0,050 | 0,066 |
| $T_C$ | 0,150 | 0,066 |
| | 0,235 | 0,042 |
| | 0,320 | 0,031 |
| | 0,405 | 0,024 |
| | 0,490 | 0,020 |
| | 0,575 | 0,017 |
| | 0,660 | 0,015 |
| | 0,745 | 0,013 |
| | 0,830 | 0,012 |
| | 0,915 | 0,011 |
| $T_D$ | 1,000 | 0,010 |
| | 1,094 | 0,008 |
| | 1,188 | 0,007 |
| | 1,281 | 0,006 |
| | 1,375 | 0,005 |
| | 1,469 | 0,005 |
| | 1,563 | 0,004 |
| | 1,656 | 0,004 |
| | 1,750 | 0,003 |
| | 1,844 | 0,003 |
| | 1,938 | 0,003 |
| | 2,031 | 0,002 |
| | 2,125 | 0,002 |
| | 2,219 | 0,002 |
| | 2,313 | 0,002 |
| | 2,406 | 0,002 |
| | 2,500 | 0,002 |
| | 2,594 | 0,001 |
| | 2,688 | 0,001 |
| | 2,781 | 0,001 |
| | 2,875 | 0,001 |
| | 2,969 | 0,001 |
| | 3,063 | 0,001 |
| | 3,156 | 0,001 |
| | 3,250 | 0,001 |
| | 3,344 | 0,001 |
| | 3,438 | 0,001 |
| | 3,531 | 0,001 |
| | 3,625 | 0,001 |
| | 3,719 | 0,001 |
| | 3,813 | 0,001 |
| | 3,906 | 0,001 |
| | 4,000 | 0,001 |

**Figure 6-9: parameters and points of the spectrum of vertical component for the SLV**

# 7 GEOTECHNICAL DESIGN APPROACH

The present chapter describes the foundation design approach as per Italian rule “NTC 2008” and Eurocode.

## 7.1 Regulatory references

- D.M. 14/01/2008 – “Norme tecniche per le costruzioni”
  Technical Rules for Construction Minister Decree (hereinafter NTC2008);

- Circ. Min. n. 617 Febbraio 2009 – “Istruzioni per l’applicazione delle Nuove norme tecniche per le costruzioni di cui al D.M. 14/01/2008”, indicata con Circ. NTC2008
  Circ. Min. n. 617 February2009 “Instructions for application of NTC2008”;

- Eurocodice 7 – “Progettazione geotecnica – Parte 1 – regole generali” nella versione in lingua italiana, pubblicata a cura dell’UNI (UNI ENV 1997-1, ratificata in data Ottobre 1994)
  EN 1997-1 (2004) (English): Eurocode 7: Geotechnical design - Part 1: General rules [Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC]”

- Eurocodice 8 – “Progettazione delle strutture per la resistenza sismica” - Parte 1: Regole generali, azioni sismiche e regole per gli edifici.
  EN 1998-1 (2004) (English): Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings [Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC

## 7.2 Ultimate Limit State (SLU)

The Ultimate Limit State (SLU) approach foresees that the following condition is respected:

$$E_d \leq R_d$$

where Ed is the design action value

$$E_d = E\left[\gamma_F F_k; \frac{X_k}{\gamma_M}; a_d\right] \quad \text{or} \quad E_d = \gamma_E \cdot E\left[F_k; \frac{X_k}{\gamma_M}; a_d\right]$$

with $\gamma_E = \gamma_F$,

while $R_d$ is the design resistance of the geotechnical system

$$R_d = \frac{1}{\gamma_R} R\left[\gamma_F F_k; \frac{X_k}{\gamma_M}; a_d\right]$$

The impact of actions and resistance is expressed as a function of the design actions $\gamma_F Fk$, of the design parameters $X_k/\gamma_M$ and of the design geometry $a_d$. The impact of the actions may also be directly assessed as $E_d=E_k\cdot\gamma_E$. In the expression of resistance $R_d$ a coefficient $\gamma_R$ explicitly appears which directly operates on system resistance.

The verification of the above condition must be done for different conditions with various safety coefficients applied on actions (A1 and A2), on resistance (R1, R2 e R3) and on geotechnical parameters (M1 and M2).

The partial safety coefficients are selected from two distinct, and alternative, design approaches:

the Approach 1 involves two different combinations of coefficient group where the first combination is generally stricter towards the structural dimensioning of the structures interacting with ground, while the second one is stricter with regards to geotechnical dimensioning;

the Approach 2 implies only one combination of coefficient groups which must be used both in the structuranl and geotechnical verifications.

Then the Ultimate Limit State verifications are the following:

- Approach 1 – Combination 1: A1 + M1 + R1
- Approach 1 – Combination 2: A2 + M2 + R2
- Approach 2: A1+ M1+R3

where A1 is for structural actions and A2 is for geotechnical actions.

The ultimate limit states for shallow foundations foresee different failure modes:

**EQU** – loss of equilibrium (of ground, structure or the set ground-structure considered as a rigid body);

**STR** – structural failure;

**GEO** – reach of limit soil resistance – bearing failure.

**UPL –** loss of structure equilibrium due to water uplift pressure

**HYD –** erosion or seepage effects

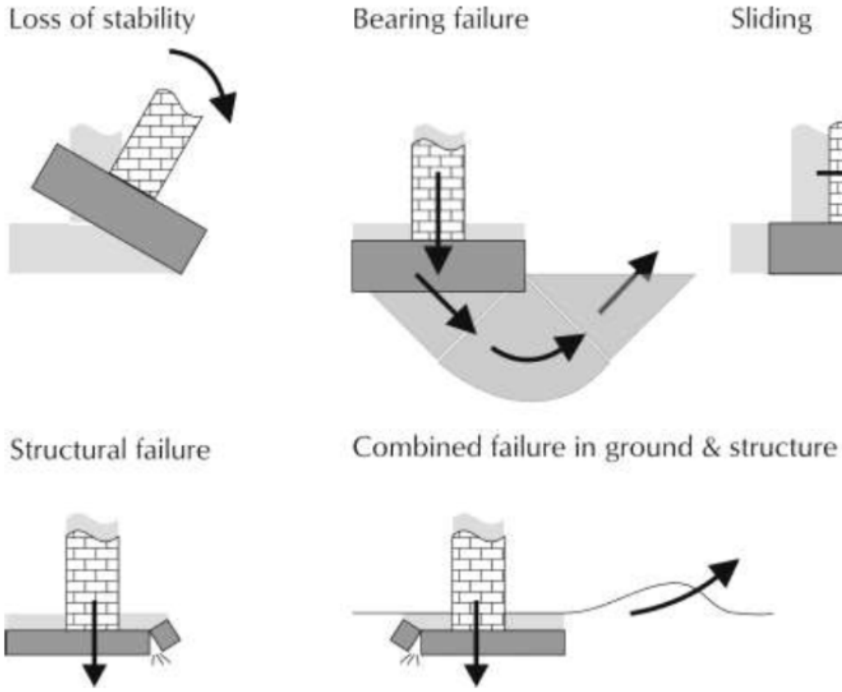



**Figure 7-1: Sketches of limit states (from Bond &Harris, 2008)**

In the present study only the failure mode GEO has been analyzed, being other failure modes not applicable. Therefore the two following Design Approaches have been used being more conservative:

Approach 1: Combination 2: (A2+M2+R2)

Approach 2: (A1+M1+R3).

In the following tables the coefficients applied on the verification are shown:

**Table 7-1 – Partial coefficients on actions (Table 6.2.I of NTC2008)**

| CARICHI | EFFETTO | Coefficiente Parziale $\gamma_F$ (o $\gamma_E$) | EQU | (A1) STR | (A2) GEO |
|---|---|---|---|---|---|
| Permanenti | Favorevole | $\gamma_{G1}$ | 0,9 | 1,0 | 1,0 |
| | Sfavorevole | | 1,1 | 1,3 | 1,0 |
| Permanenti non strutturali (1) | Favorevole | $\gamma_{G2}$ | 0,0 | 0,0 | 0,0 |
| | Sfavorevole | | 1,5 | 1,5 | 1,3 |
| Variabili | Favorevole | $\gamma_{Qi}$ | 0,0 | 0,0 | 0,0 |
| | Sfavorevole | | 1,5 | 1,5 | 1,3 |

(1) Nel caso in cui i carichi permanenti non strutturali (ad es. i carichi permanenti portati) siano compiutamente definiti, si potranno adottare gli stessi coefficienti validi per le azioni permanenti.

**Table 7-2 – Partial coefficients on geotechnical parameters (Table 6.2.II of NTC2008)**

| PARAMETRO | GRANDEZZA ALLA QUALE APPLICARE IL COEFFICIENTE PARZIALE | COEFFICIENTE PARZIALE $\gamma_M$ | (M1) | (M2) |
|---|---|---|---|---|
| *Tangente dell'angolo di resistenza al taglio* | $\tan \varphi'_k$ | $\gamma_{\varphi'}$ | 1,0 | 1,25 |
| *Coesione efficace* | $c'_k$ | $\gamma_{c'}$ | 1,0 | 1,25 |
| *Resistenza non drenata* | $c_{uk}$ | $\gamma_{cu}$ | 1,0 | 1,4 |
| *Peso dell'unità di volume* | $\gamma$ | $\gamma_\gamma$ | 1,0 | 1,0 |

**Table 7-3 – Partial coefficients for SLU verification on shallow foundations (Table 6.4.I of NTC2008)**

| VERIFICA | COEFFICIENTE PARZIALE (R1) | COEFFICIENTE PARZIALE (R2) | COEFFICIENTE PARZIALE (R3) |
|---|---|---|---|
| Capacità portante | $\gamma_R = 1,0$ | $\gamma_R = 1,8$ | $\gamma_R = 2,3$ |
| Scorrimento | $\gamma_R = 1,0$ | $\gamma_R = 1,1$ | $\gamma_R = 1,1$ |

## 7.3 Limit Load for collapse of soil foundation

For the calculation of the collapse load limit of the set foundation-soil we will proceed to evaluate the bearing capacity limit of the ground (failure load) by referring to traditional methods based on the limit equilibrium theory as originally proposed by Brinch- Hansen (1970) for homogenous soil.

$$q_{lim} = \frac{1}{2}\gamma' \cdot B' \cdot N_\gamma \cdot s_\gamma \cdot d_\gamma \cdot i_\gamma \cdot b_\gamma \cdot g_\gamma + c' \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot b_c \cdot g_c + q' \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q$$

where:

- $q_{lim}$ = limit bearing capacity;
- γ` = effective soil weiht;
- $z_w$ = distance between ground water level and base of foundation level;
- B' = lenght of smaller side of effective equivalent foundation;
- $N_\gamma, N_c, N_q$ = bearing capacity coefficients, depending on effective friction angle $\phi'$;
- $s_\gamma, s_c, s_q$ = shape factors;
- $d_\gamma, d_c, d_q$ = foundation base level factors;
- $i_\gamma, i_c, i_q$ = corrective factors for inclined loads;
- $b_\gamma, b_c, b_q$ = corrective factors for inclined base foundation;

- $g_\gamma, g_c, g_q$ = corrective factors for inclined ground surface;
- q' = laterale upload at the base of foundation.

$\phi'$ is defined as "design friction angle "$\phi'_d$ obtained from the characteristic friction angle $\phi'_k$ multiplied for the partial coefficient M1 or M2, of the geotechnical parameters.

The above factors are defined by the following expressions:

- Bearing Capacity Coefficient (Vesic, 1975)

$$N_\gamma = 2 \cdot (N_q + 1) \cdot \tan\phi'$$

$$N_q = e^{(\pi \cdot \tan\phi')} \cdot (\frac{\pi}{4} + \frac{\phi'}{2})$$

$$N_c = (N_q - 1)\frac{1}{\tan\phi'}$$

- foundation shape factors (De Beer, 1967)

$$s_\gamma = 1 - 0.4\frac{B'}{L'}$$

$$s_q = 1 + \frac{B'}{L'} \cdot \tan\phi'$$

$$s_c = 1 + \frac{B'}{L'} \cdot \frac{N_q}{N_c}$$

where L' = lenght of longer side of effective equivalent foundation

- foundation base level factors (Brinch-Hansen, 1970)

$$d_\gamma = 1$$

$$d_q = 1 + 2 \cdot \frac{D}{B'} \cdot \tan\phi' \cdot (1 - \sin\phi')^2$$ , per D/B'≤ 1

$$d_q = 1 + 2 \cdot \tan\phi' \cdot (1 - \sin\phi')^2 \cdot \tan^{-1}(\frac{D}{B'})$$ , per D/B'≥ 1

$$d_c = d_q - \frac{1 - d_q}{N_c \cdot \tan\phi'}$$

where D = depth of foundation base below ground level

- corrective factors for inclined loads (Vesic, 1975)

$$i_\gamma = \left[1 - \frac{H}{N + B' \cdot L' \cdot c' \cdot \cot\phi'}\right]^{(m+1)}$$

$$i_q = \left[1 - \frac{H}{N + B' \cdot L' \cdot c' \cdot \cot\phi'}\right]^{m}$$

$$i_c = i_q - \frac{1 - i_q}{N_q - 1}$$

$$m_B = \frac{2 + B' / L'}{1 + B' / L'}$$

$$m_L = \frac{2 + L' / B'}{1 + L' / B'}$$

$$m = m_L \cdot \cos^2\vartheta + m_B \cdot \sin^2\vartheta$$

$$\vartheta = arctg(\frac{T_B}{T_L})$$

where H = horizontal load

N = vertical load.

$T_B$ = horizontal load in direction of B

$T_L$ = horizontal load in direction of L

- corrective factors for inclined base foundation (Brinch-Hansen, 1970)

$$b_q = b_\gamma = (1 - \alpha \cdot \tan\phi')^2$$

$$b_c = b_q - \frac{1 - b_q}{N_c \cdot \tan\phi'}$$

where α = inclination of foundation base;

- corrective factors for inclined ground surface (Brinch-Hansen, 1970)

$$g_q = g_\gamma = (1 - \tan\omega)^2$$

$$g_c = g_q - \frac{1 - g_q}{N_c \cdot \tan\phi'}$$

where $\omega$ = inclination of ground surface.

The effective dimensions of the equivalent foundation B' and L' are evaluated, in case of eccentric loads, using Meyerhof criteria that suggests to use the net area (B',L') for the calculation of bearing capacity:

$B' = B - 2e_1$

$L' = L - 2e_2$

where:

L, B = "actual" dimensions of foundation;

$e_1, e_2$ = eccentricity of load in considered directions.

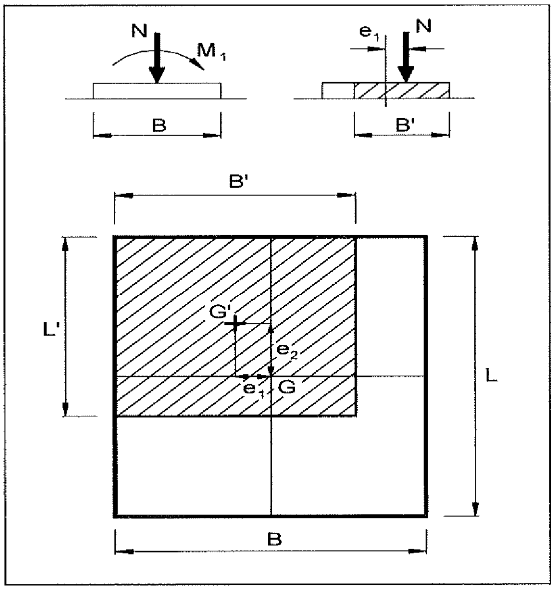



**Figure 7-1: Meyerhof criteria for the calculation of rearing foundation**

In order to consider the groundwater level effect on the ground below the foundation, the unit volume weight ($\gamma_c$) as follows:

$$\gamma_c = \gamma_W (z_W / B) + (\gamma - \gamma_W) \text{ se } 0 \le z_W \le B$$

$$\gamma_c = \gamma \text{ se } z_W \ge B$$

where

$z_W$ = distance between foundation base and groundwater level.

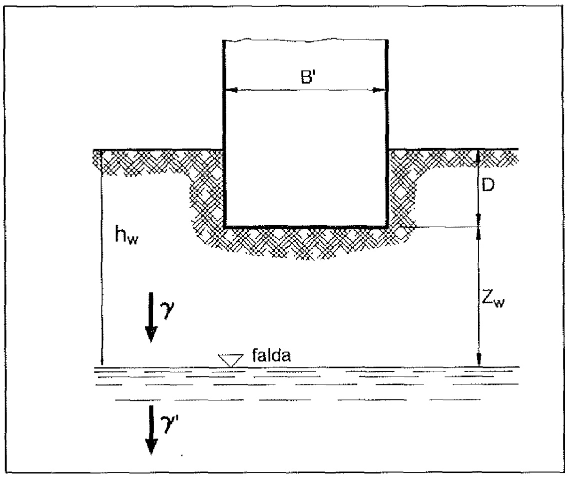



**Figure 7-2: Influence of groundwater**

## 7.4 Serviceability limit state (SLE)

NTC 2008 indicates to calculate the values of displacements and distortions to verify its compatibility with the performance requirements of the structures in elevation (§§ 2.2.2 and 2.6.2 of the NTC 2008), in compliance with the condition (§ 6.2.7 of NTC):

$E_d \leq C_d$

where $E_d$ is the design value of the actions and $C_d$ is the prescribed limit of the effect of actions.

Settlements assume generally different values on the laying surface of a building. Then it is necessary the estimation of differential settlements ,i.e. the settlement difference among points of a same foundation, of foundations with common superstructures and with statically independent superstructures. In this case the estimation of direct settlements for a plinth subjected to a vertical load will be carried out, while differential settlements between plinths or mutual settlements produced by plinths positioned in proximity will not be examined.

The values of the mechanical properties to be used in the analysis are those characteristic and partial factors on actions and on the strength parameters are set equal to 1.0.

The calculation of direct settlement is carried out in the elastic theory, by means of the correlation envisaged by Davis and Poulos (1974), which provides for the calculation of the stress state induced in the soil, supposing a linear elastic semi-space homogeneous and isotropic, characterized for each i-th layer from the elastic modulus ($E_i$) and the coefficient of Poisson ($\nu$). This makes possible to take into account the soil stratigraphy layer change with depth. The procedure steps are:

a) σx, σy e σz versus depth z are calculated by means of following equations:

$$\sigma_z = \frac{q}{2\cdot\pi}\cdot\left[\arctan\left(\frac{L\cdot B}{z\cdot R_3}\right)+\frac{L\cdot B\cdot z}{R_3}\cdot\left(\frac{1}{R_1^2}+\frac{1}{R_2^2}\right)\right]$$

$$\sigma_x = \frac{q}{2\cdot\pi}\cdot\left[\arctan\left(\frac{L\cdot B}{z\cdot R_3}\right)-\frac{L\cdot B\cdot z}{R_1^2\cdot R_3}\right]$$

$$\sigma_y = \frac{q}{2\cdot\pi}\cdot\left[\arctan\left(\frac{L\cdot B}{z\cdot R_3}\right)-\frac{L\cdot B\cdot z}{R_2^2\cdot R_3}\right]$$

where:

q = load applied on the foundation;

B = smaller side of the foundation;

L = larger side of the foundation.

$$R_1 = \left(L^2 + z^2\right)^{0,5}$$

$$R_2 = \left(B^2 + z^2\right)^{0,5}$$

$$R_3 = \left(L^2 + B^2 + z^2\right)^{0,5}$$

b) Then the distribution of the vertical deformation is determined along the depth z, considering the layered soil with E and ν for each layer:

$$\varepsilon_z = \frac{\sigma_z}{E_i} - \frac{\upsilon_i}{E_i}\cdot(\sigma_x + \sigma_y)$$

c) Then the settlements are calculated by integration of the vertical deformation along the depths:

$$\delta = \int_0^H \varepsilon_z \cdot dz$$

where H is the ground stratum which settlements are calculated for at depth z where the two conditions are both verified:

$$H \geq 2\cdot B \qquad \sigma_z(H) \leq 0.15\cdot\sigma_{z0}$$

with

$\sigma_z(H)$ = lateral vertical load applied beside the foundation base

$\sigma_{z0}$ = geostatic stress state.

# 8 BEARING CAPACITY VERIFICATIONS

The main structures ofr the pipeline route to be verified with regard to bearing capacity are the pipe saddles and the slab for the Block Valve Station (BVS).

The verification of the bearing capacity has been performed providing abacus diagrams with the following foundation base geometry:

- pipe saddle 1 x 1 m
- slab 8 x 8 m

The base of foundation has been assumed at -2.50 m b.g.l., and the beside ground level is assumed the 0,0 bgl corresponding to a lateral load of about 25 kPa. The watertable has been assumed at 2,30 m bgl as provided by piezometer readings in the zone close to the BVS.

The bearing capacity (Design resistance) has been calculated using the two design approaches applying the partial coefficients on geotechnical parameters and resistances as per NTC2008.

- Approach 1: Combination 2: A2+M2+R2
- Approach 2: A1+M1+R3

The results of the calculation have been provided by means of diagrams/abacus, where the x-axis is the eccentricity in direction of B and L, the y-axis is the design resistance Rd [kN].

Each diagram is valid for a single set of foundation geometry (n.2 sets) and for a single characteristic friction angle $\phi'_k$ Three curves are drawn for three load ratio values N/H between Vertical load and Horizontal Load (N/H = 0%, 5%, 10%;) along both directions B and L.

The verifications have been analyzed assuming conservatively that the soil unit interested by the foundations is the unit b1. Therefore one single set of friction angle have been analyzed using $\phi'_k$ = 29,5°, corresponding to a design friction angle $\phi'_d$ =24,4°.

The structural designer can enter the applied load (multiplied for the relative coefficient) and actual eccentricity and evaluate which geometry satisfy the load requirements, or vice versa, fixed the geometry, determine the allowable load that can be applied.

Regarding the slope of the sides of the excavation for burying the pipeline, it depends on the soil lithology.

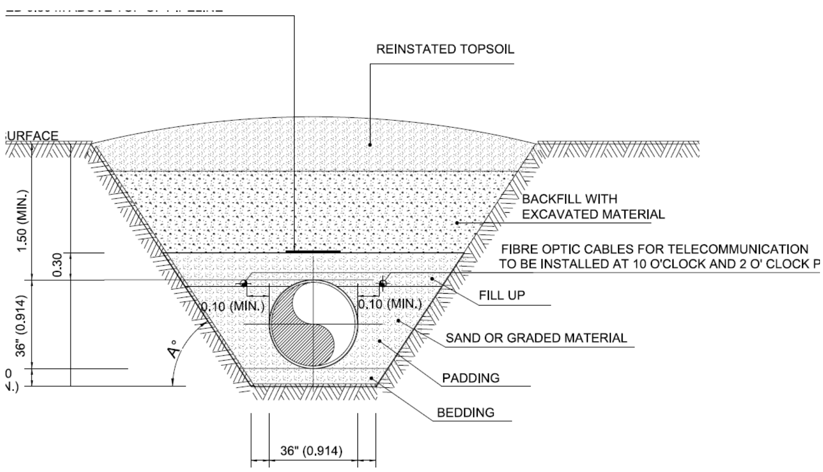


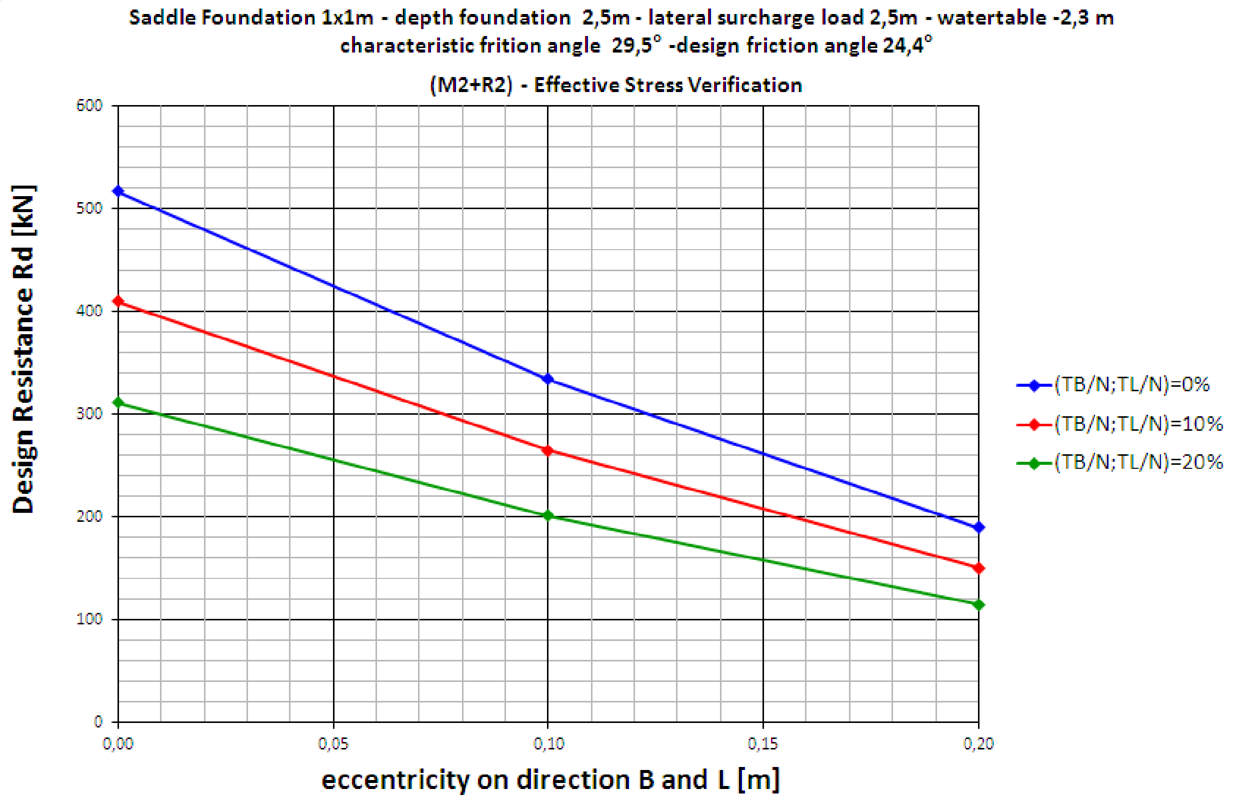

**Figure 8-1: Typical onshore pipeline trench** *(source document: IPL00-SPF-100-F-DFT-0002_01)*

The field results confirm the proposed slope of excavation sides (called A° in the above sketch) as follows:

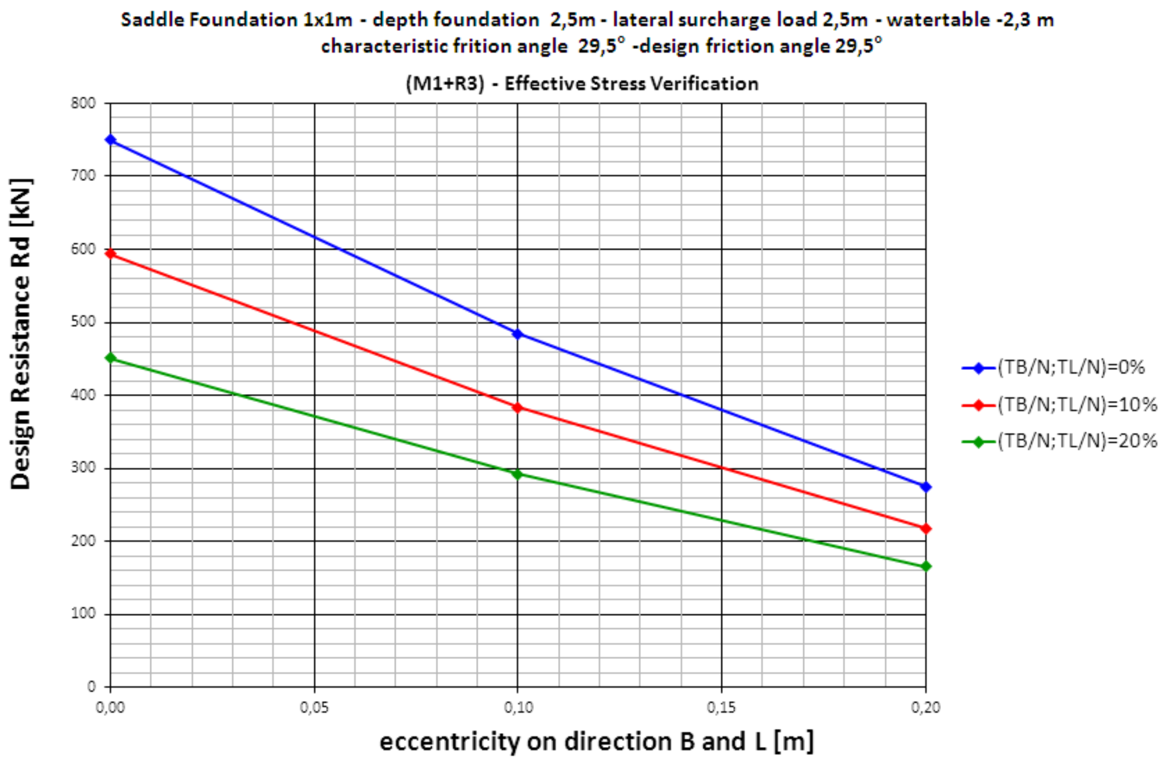
- 80° in rock media
- 60° in case of cohesive layers
- 45° for non cohesive soil and/or cohesive soil moderately consistent.

Special care should be taken into account for the excavation in case of groundwater at the bottom of the trench in order to avoid local earth-falls. Excavation may be carried out without retaining structures only in case of rock, anyway dewatering the trench..

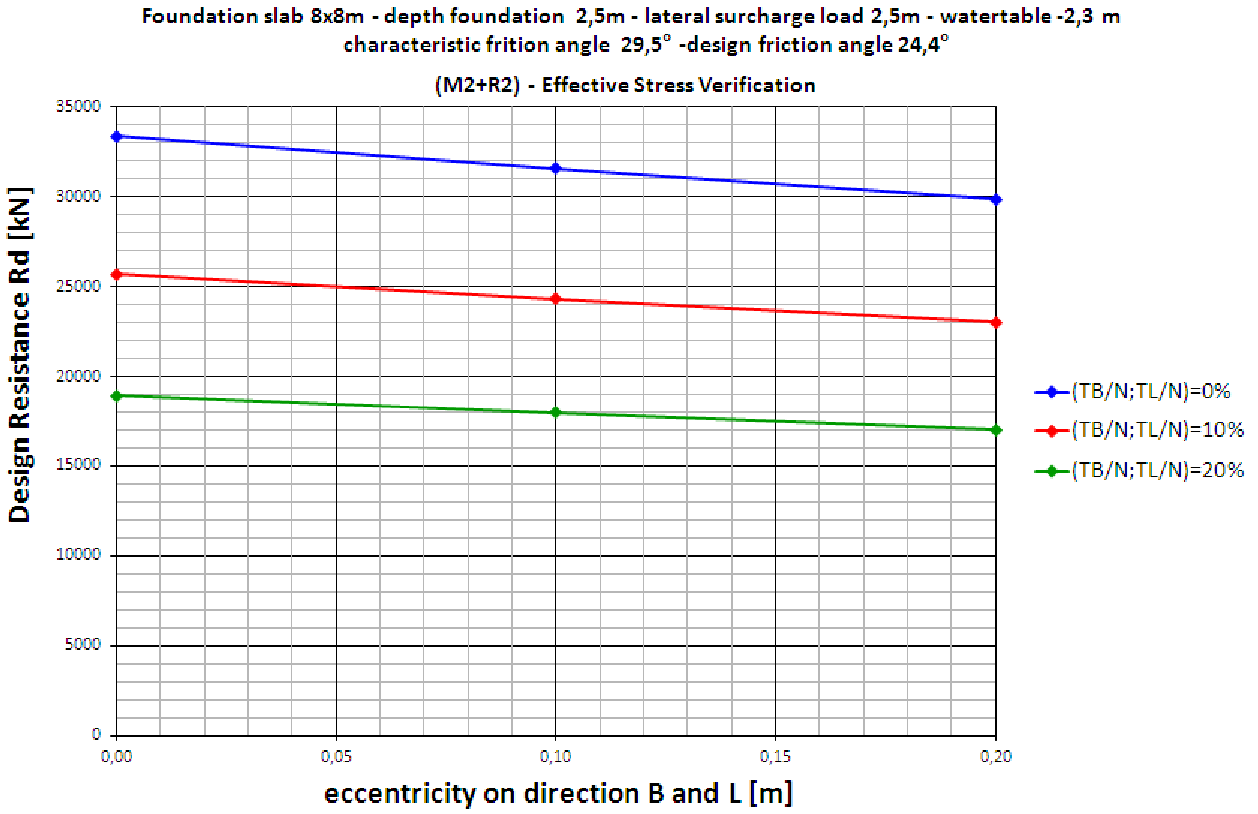
## 8.1 Saddle Foundation 1x 1 m- Verification M2+R2



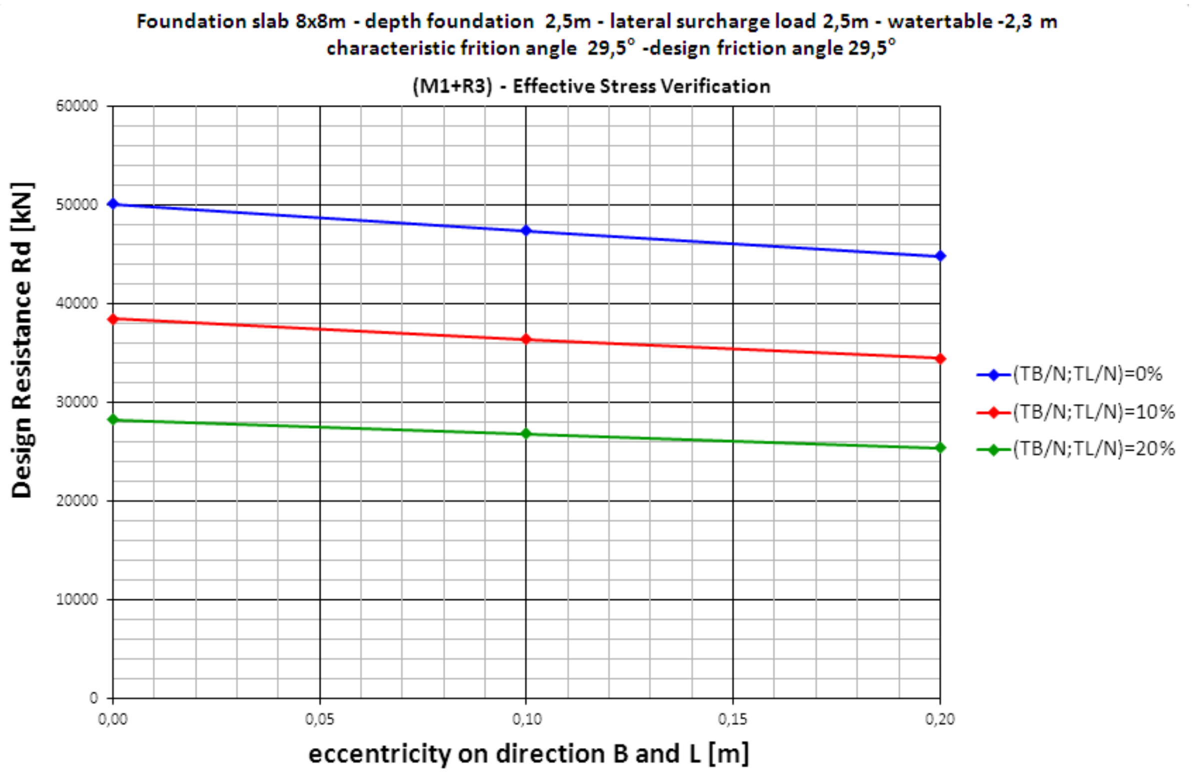

## 8.2 Saddle Foundation 1x 1 m- Verification M1+R3



## 8.3 Foundation slab 8 x 8 m- Verification M2+R2




## 8.4 Foundation slab 8 x 8 m- Verification M1+R3



# 9 SETTLEMENTS VERIFICATION

The settlements verifications are provided for the same geometries analyzed for the bearing capacity verifications as follows:

- pipe saddle 1 x 1 m
- slab 8 x 8 m

The base of the foundation was assumed at 2.50 m bgl, and the ground level at 0 m bgl (corresponding to a lateral load of about 10 kPa. The water table was taken at about 4 m bgl as determined by borehole ST_ BH2.

Young's Modulus has been provided with the three values: one derived from the statistic data analysis (E' = 16 MPa) and two other values coming from PRT baseline foundation characteristics values:

E' = 11 MPa and E' = 25 MPa

$\nu$ = Poisson's Modulus assumed = 0.2

The increasing stiffness of the calcarenite bedrock layer has been neglected cautelatively.

The results are reported in diagrams where the x-axis is the applied load and y-axis the expected elastic settlement for the two Young's modulus.

These abacus allows the structural engineer, once defined the final foundation geometry, to enter the vertical applied load (with the relative coefficient multiplier) and to determine the settlement that has to be verified on the serviceability limit state (SLS).

Eurocode (1) provides for a stand-alone foundation an acceptable

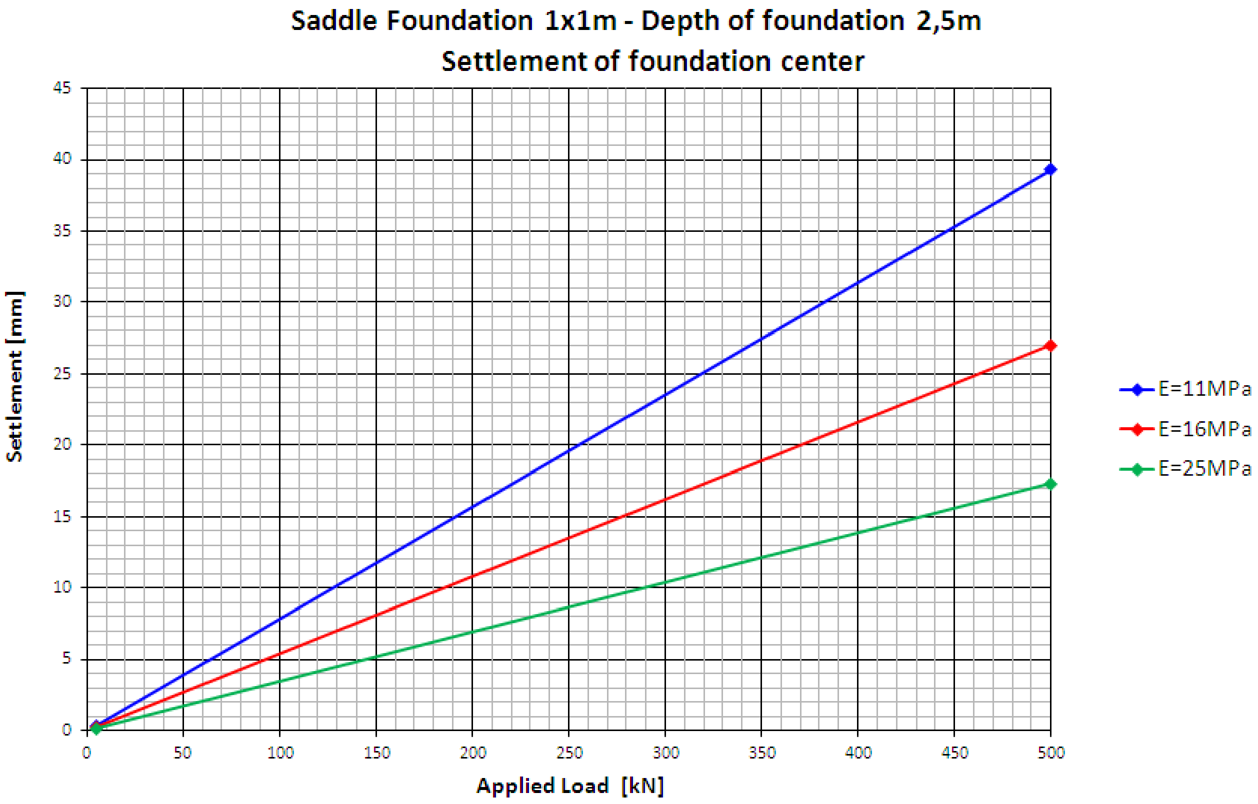
- settlement equal to $s_{max} \leq 25$ mm
- rotation between: $\beta_{max}$= 1/300 ÷ 1/2'000.

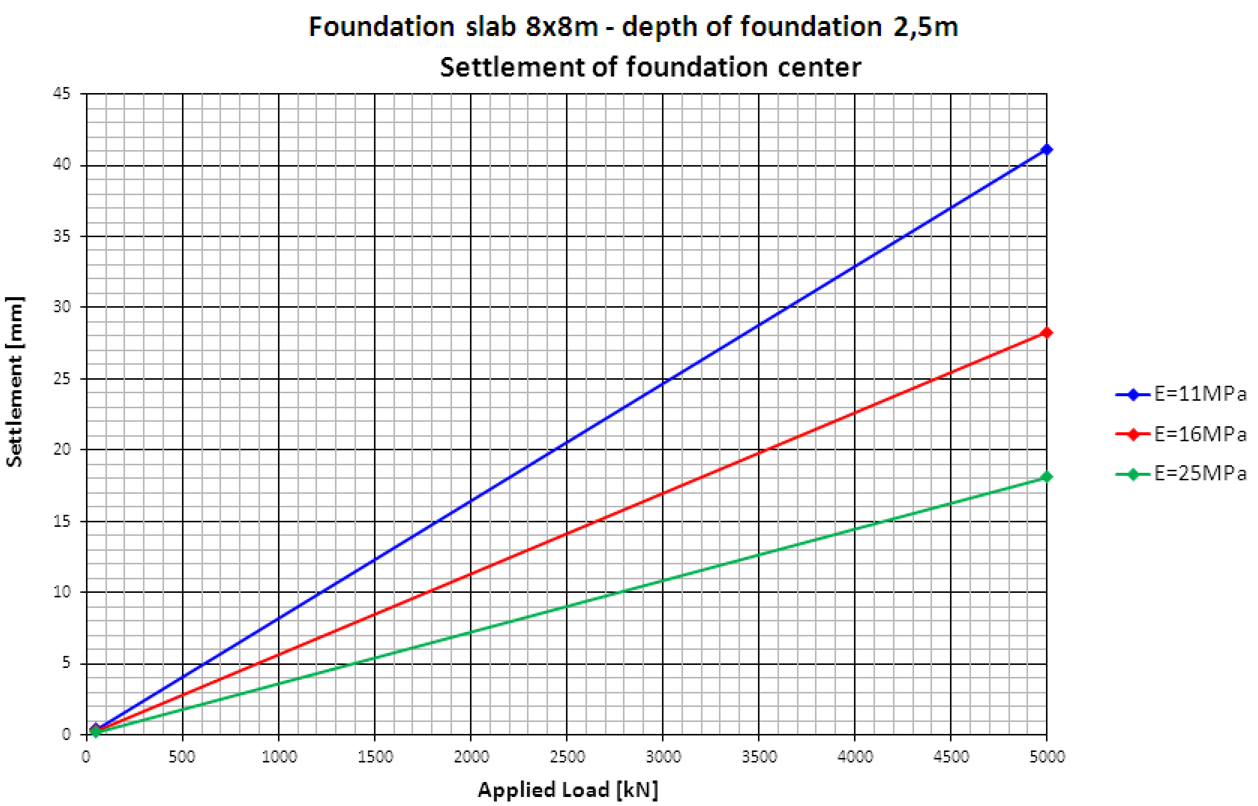
Notwithstanding, Eurocode allows settlements up to 50 mm in case of framed buildings.

Entering the value of 25 mm or 50 mm in the diagrams y-axis, the applied load is then determined and has to be compared with actual design load.

The following diagrams provide the settlement of the centre of the foundation for the various geometries.

[1] *(ref. 2013) - Shallow foundations -G. Scarpelli and T.L.L.Orr - Worked examples presented at the Workshop "Eurocode 7: Geotechnical Design"- Dublin, 13-14 June, 2013 -Support to the implementation, harmonization and further development of the Eurocodes*


Saddle Foundation 1x1m - Depth of foundation 2,5m
Settlement of foundation center
Settlement [mm]
Applied Load [kN]
0
5
10
15
20
25
30
35
40
45
50
100
150
200
250
300
350
400
450
500
E=11MPa
E=16MPa
E=25MPa


Foundation slab 8x8m - depth of foundation 2,5m
Settlement of foundation center
Settlement [mm]
Applied Load [kN]
0
5
10
15
20
25
30
35
40
45
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
E=11MPa
E=16MPa
E=25MPa

# 10 RECOMMENDATIONS

11 road crossings are present along the pipeline route, as reported in tab. 2-1. Excluding road crossings 1 and 2 (intersected by the microtunnel), 9 are left, including 3 in the vicinity of boreholes (BH4, BH9, BH11) located from 0 to 5 m away from the road.

The possibility of further geotechnical investigations in the order of 2 boreholes each road crossing (1 in the cases of the three road crossings above mentioned) down to the design crossing depth should be checked, while also considering to implement 2/3 SPT testing and / or samples collection each borehole for analysis in the geotechnical laboratory, on the basis of whose results the most appropriate method of under-crossing will be chosen, along with other criteria such as logistics, traffic flow, etc.

According to the workability classes of the standard DIN 18300 “Earthwork”, soils encountered along the pipeline route may be classified as it follows:

- class n° 6 *(rock which is easy to excavate and similar soils; more than 30 % stones with each 0,01 to 0,1 $m^3$; solid clay and silt)*);
- class n° 7 *(rock which is hard to excavate)*.

Once the piping is laid, trench shall be refilled. It is recommended that filling material has similar geotechnical properties to those of the surrounding natural ground. In order to do that, it shall be compacted with a manual compactor up to 50 cm above the piping subsequently superimposing successive layers of filling material 30 cm thick each.

Diversi test dovranno essere effettuati in una sezione di trincea di prova al fine di verificare il materiale di riempimento e il sistema di compattazione, nonché i valori di permeabilità raggiungibili.

Several test shall be performed in a section of the test trench in order to verify the filling material, the compaction system as well as the reachable hydraulic conductivity values.

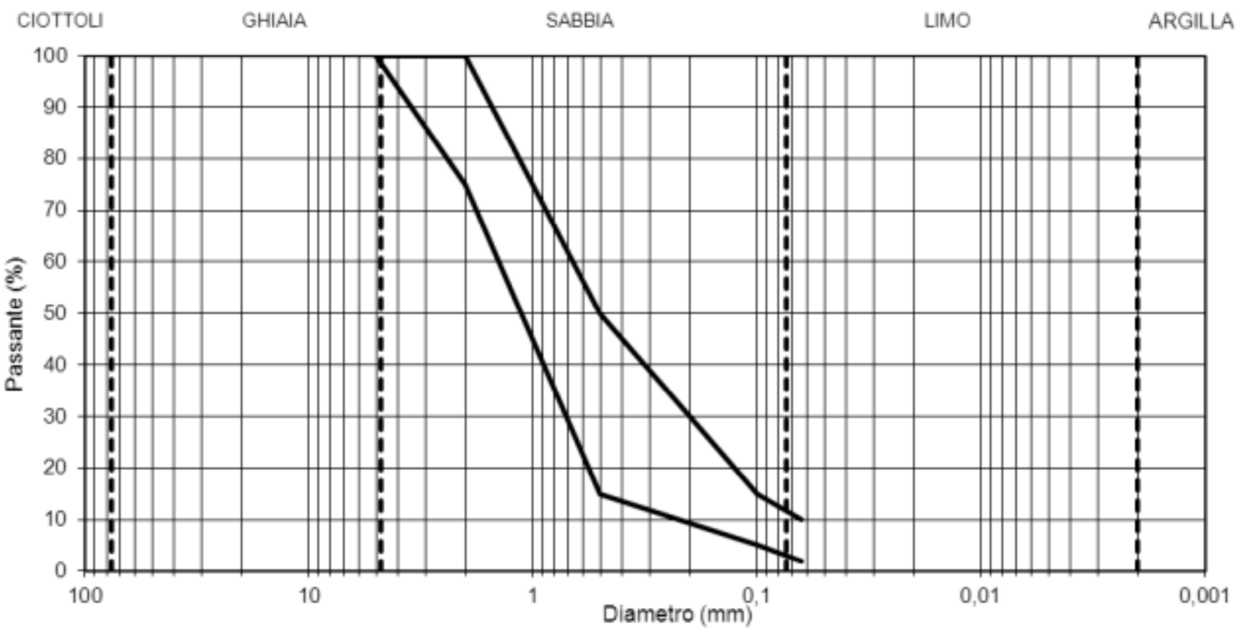
It is suggested to reuse as much as possible excavated soil like filling material for the trench, in compliance with what is provided by current regulation (Legislative Decree 152/2006 and subsequent amendments and Ministerial Decree 161/2012) which requires the preparation of a specific document, Piano di Utilizzo Terre, containing all the indications for the excavated soils managements (reuse, disposal, etc.).

In case of selection of material from quarries, referring to the classification UNI 10006 shown in the following table, this material shall be class A2-A3, with less than 35% passing through a sieve of 0,0075 mm (n. 200 sieve).

## Table 10-1 – Classification UNI 10006

| Classificazione generale | Terre ghiaio-sabbiose<br>Frazione passante allo staccio<br>0.063 mm ≤ 35% | | | | | | | Terre limo-argillose<br>Frazione passante allo staccio<br>0.063 mm > 35% | | | | | Torbe e terre organiche palustri |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Gruppo | A1 | | A3 | A2 | | | | A4 | A5 | A6 | A7 | | A8 |
| Sottogruppo | A1-a | A1-b | | A2-4 | A2-5 | A2-6 | A2-7 | | | | A7-5 | A7-6 | |
| Frazione passante allo staccio<br>2 mm<br>0.4 mm<br>0.075 mm | ≤ 50<br>≤ 30<br>≤ 15 | -<br>≤ 50<br>≤ 25 | -<br>> 50<br>≤ 10 | -<br>≤ 35 | -<br>≤ 35 | -<br>≤ 35 | -<br>≤ 35 | -<br>> 35 | -<br>> 35 | -<br>> 35 | -<br>> 35 | -<br>> 35 | |
| Caratteristiche della frazione passante allo staccio 0.4 mm<br>LL (Limite liquido)<br>IP (Indice di plasticità) | -<br>≤ 6 | -<br>≤ 6 | -<br>N.P. | ≤ 40<br>≤ 10 | > 40<br>≤ 10 | ≤ 40<br>> 10 | > 40<br>> 10 | ≤ 40<br>≤ 10 | > 40<br>≤ 10 | ≤ 40<br>> 10 | > 40<br>> 10<br>IP ≤ LL-30 | > 40<br>> 10<br>IP > LL-30 | |
| Indice di gruppo | 0 | | 0 | 0 | | ≤ 4 | | ≤ 8 | ≤ 12 | ≤ 16 | ≤ 20 | | |
| Tipi usuali dei materiali caratteristici costituenti il gruppo | Ghiaia o breccia, ghiaia o breccia sabbiosa, sabbia grossa, pomice, scorie vulcaniche, pozzolane | | Sabbia fina | Ghiaia o sabbia limosa o argillosa | | | | Limi poco compres-sibili | Limi molto compres-sibili | Argille poco compres-sibili | Argille molto compres-sibili e media-mente plastiche | Argille molto compres-sibili e molto plastiche | Torbe di recente o remota formazione, detriti organici |
| Qualità portanti quale terreno di sottofondo in assenza di gelo | da eccellente a buono | | | | | Da mediocre a scadente | | | | | | | Da scartare |
| Azione del gelo sulle qualità portanti | Nessuna o lieve | | | Media | | | | Molto elevata | | Media | Elevata | Media | |
| Ritiro e rigonfiamento | Nullo | | | Nullo o lieve | | | | Lieve o medio | | Elevato | Elevato | Molto elevato | |
| Permeabilità | Elevata | | | Media o scarsa | | | | | | Scarsa o nulla | | | |

Here below is the proposed particle size distribution of the filling material.



| Sieve (mm) | % passing, fuse inferior limit | % passing, fuse superior limit |
|---|---|---|
| 5 | 100 | 100 |
| 2 | 75 | 100 |
| 0,5 | 15 | 50 |
| 0,1 | 5 | 15 |
| 0,063 | 2 | 10 |

**Figure 10-1: Proposed grain size distribution for tests**

Once the proper filling material will have been defined, it is suggested the verification and the check in progress of the filling of the trench by the means of the following tests to be performed each 250/300 m:

a) Lefranc tests

b) Proctor tests

c) In situ density tests

Lefranc tests will be the most important to determine the local hydraulic conductivity of the filling and will be carried at about 2.5 m of depth within the trench from the side of the duct just compacted. Given the characteristics of the soil, the hydraulic conductivity values must be within the following range: between $K_{fill}$ = 4.0 × 10-4 m / s and 5.0 × 10-5 m / s.

Lefranc tests are performed during the progress of the drilling in non-rocky soils. The test is performed by measuring the water absorption in the ground, filtering the water through a predetermined section of the hole.

In case of high conductivity of the ground, the test is performed with variable hydraulic load, while in case of low-medium conductivity with constant hydraulic load. The Italian rules and specifications shall be: - AGI Italian Geotechnical Association - (1977) - Recommendations on design and implementation of Geotechnical Investigations.

The test method shall be as follows:

- rotary coring drilling down to the depth of the test (in this case about 2.5 m bgl)
- casing of the borehole down to the depth reached
- raising up of the casing of 1 m
- measurement of the groundwater level in the borehole (if present) repeated several times
- performance of the test, according to the following criteria:
  - ❖ VARIABLE LOAD TEST
    - ✓ Fill with water until the end of the coating.
    - ✓ Measurement of the water level inside the tube in a time interval of 15 '', 30 '', 1 ', 2', 4 ', 8', 15 'from the beginning of the test.

- CONSTANT LOAD TEST
  - ✓ pouring of clean water into the borehole until the hydraulic load becomes constant corresponding to a time-constant and measured flow absorption rate of the soil.
  - ✓ Check of the flow using a calibrated flowmeter with a sensitivity of 0.1 l / min.
  - ✓ The release of water must be kept with a constant flow rate without any modification, for 10-20 minutes.
  - ✓ Starting from the end of the flow of water, measurement of the progressive lowering of the water level inside the tube will be performed, in a time interval of 15 '', 30 '', 1 ', 2', 4 ', 8', 15 ', continuing until reaching the constant water level.

Proctor test is a test method used to determine the properties of compaction of the ground, particularly the maximum density obtainable by compaction of the dry fraction of the soil and the corresponding moisture content, called "optimum moisture". The original test is the Standard Proctor Test which has since been amended as Modified Proctor Test (ref. "Standard Test Methods for Laboratory Compaction Characteristics of Soil" ASTM D698 and ASTM D1557).

The test consists in constipating soil samples with a given water content and standard compaction energy. The soil is previously dried and divided into 4 to 6 samples. The moisture content of each sample is adjusted by adding water (increases 3% - 5% or more based on the type of soil).

The soil is placed in a cylinder of 4-inch in diameter in three different layers each of which is compacted receiving 25 strokes of a 2.5 kg pestle falling from a height of 30.5 cm. Before adding each new level, the surface of the previous layer is scraped to ensure a uniform distribution of the effects of compaction.

At the end of the test, after removing and drying the sample, dry density and water content for each Proctor Test are measured. With the values obtained the curve (curve of thickening) of the weight of dry volume (or density) as a function of the water content corresponding is plotted in the laboratory. From this curve the optimal water content corresponding to the maximum value of the weight of dry volume (dry density) is determined.

The difference between the two tests (Standard and Modified) consists mainly in the energy compaction.

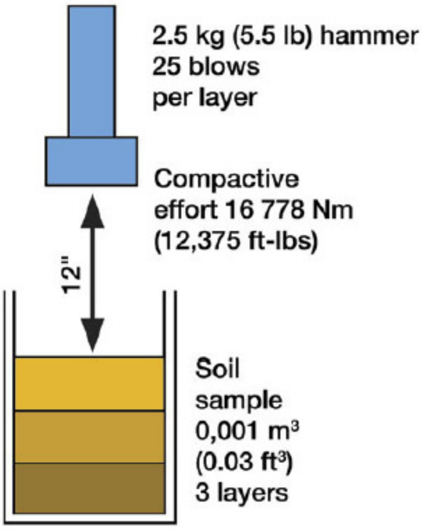



**Figure 10-2: Sketch for Proctor test**

The density test site is used to determine the density and the water content of both the compacted and natural soils having particles with size <20 mm. The test consists in filling a hole of known volume with calibrated sand, whose density is determined using a cylinder of known volume, equal to that of the test hole. The reference standard is ASTM D 1556-90 (Standard Test Method for Density and the unit weight of soil on site by the method Sand-Cone).

The central hole of the metal plate is placed on a suitably leveled surface of the ground using the central hole as a shape. The soil is dug by means of a shovel to the desired depth and the loose material is carefully removed and collected in the metallic container and is weighed = W.

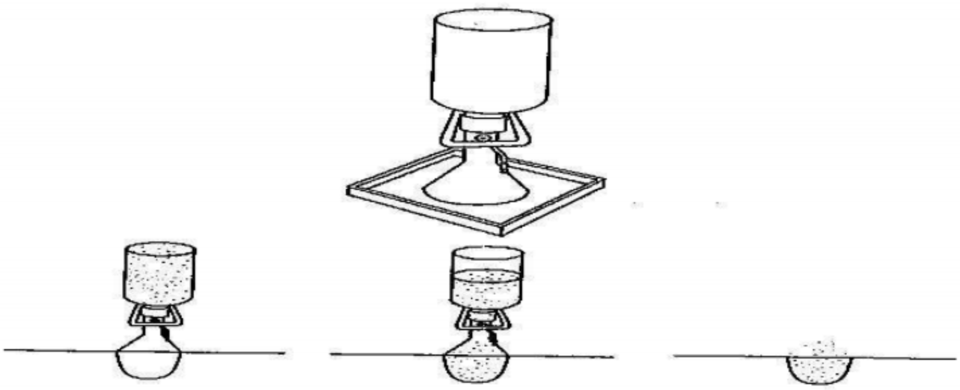


**Figure 10-3: sketch for site density test**

The metal plate with a central hole is removed and the sand cylinder is positioned centrally on the hole. The shutter is opened to let the sand get off completely, by gravity, into the hole and the retaining cone until there is no further movement of sand in the cylinder. Then the shutter is closed and the cylinder is weighed again to determine the weight W4 = volume of sand that fills the hole = Wb.

The moisture content of the soil dug w% is determined by taking a soil sample, weighing it, drying it in a stove at 110 ° C and weighing it again, or alternatively by placing the entire excavated soil (weight W) in the oven and finding its dry weight = Wd.

The sand filling of the hole and the samples for moisture determination are weighed at least three times and average values are used for the determination of the on site density (wet and dry).

Calculation and results:

W1 = weight of the cylinder filled with sand up to 10 mm from the upper edge.

W2 = weight of the sand contained in the cone

W3 = weight of the cylinder and after pouring sand into the container and in the cone

W4 = weight of the cylinder and the sand after pouring into the hole dug and in the cone;

Va = volume of the container cm3

W = weight of excavated soil

Wd = dry weight of excavated soil

w = water content in the soil%

The weight of filling sand contained in the container calibration = Wa = (W1-W2-W3)

(S) contained in the bulk density of sand gs = Wa / Vs

Weight of the sand filling of the hole = Wb = (W1 -W4 -W2)

Volume of sand filling hole = V = Wb / gs

(Ii) density of moist soil dug in-situ g = W / V

(Iii) Water content of the soil w% = (100 (W - Wd)) / Wd%

(Iv) Density of dry soil dug gd gs = Wd / W)

The results are reported as the mean value of at least three series of tests as follows:

(i) the wet density of the soil in place in g / $cm^3$, rounded to the second decimal place

(ii) the dry density of the soil in place g / $cm^3$

(iii) water content of the soil as a percentage, rounded to the first decimal place.

Test results will be analyzed by the works director who may require changes to compaction in case of differences between the characteristics of the compacted material and the characteristics of the surrounding soil.

# 11 CONCLUSIONS

The present report describes the field work investigations that have been carried out along the onshore pipeline route.

The results of geophysical field tests and of borehole investigations, including field and laboratory tests) have been processed to obtain the geological, geotechnical and seismic characterization.

The soil layers encountered have been reported in the interpretive geological cross sections showing thickness and layout of the various geotechnical units. Based on geotechnical boreholes performed, some of which are very far from each others, it is not possible to exclude that different soil/rock conditions may be encountered along the whole pipeline trench, also because of the possible presence of heteropic relationships between the different lithotypes.

Where the karst caves were actually encountered during TBM excavation, it is suggested to perform an injection of cement mixtures or filling in lean concrete, according to the following phases of execution (which will then be better defined by the designer):

- Rough estimate of the size of the cavity, possibly by means of geophysical investigation during TBM excavation;
- Removal of residual water;
- Fill with mixtures based on cement or concrete thin;
- Check the system has been filled and expected stabilization after surgery.
- These recommendations are approximate and should be verified by the designers of TBM excavation.

For the excavation works, according to classes of workability of DIN 18300 “Earthwork”, classes n° 6 and n° 7 should be considered.

Regarding the slope of the sides of the excavation for burying the pipeline, the field results confirm the proposed slope of excavation sides as follows:

- 80° in rock media
- 60° in case of cohesive layers
- 45° for non-cohesive soil or cohesive soil moderately consistent.

Special care during excavation has to be considered in case of presence of groundwater close to the bottom of excavation, in order to avoid collapse of the trench sides (in case of sandy soil).

The geotechnical design approach analyzed the bearing capacity and expected settlements in the hypothesis of shallow foundations, on the basis of structural drawings provided. Two foundation types have been verified, saddle foundation and a foundation slab for the Block Valve Station. having foundation depth is at 2.5 m b.g.l. interesting the geotechnical unit b1

In case of undercrossing of existing intersecting roads, the underpass techniques for pipe installation should be verified considering the soil conditions locally, and eventually require a local dedicated investigation.

The use of material with similar geotechnical properties to those of the on-site natural soil is recommended for the refilling of trench. For this purpose it is suggested to re-use as much as possible the excavated material, according to the regulations in force (Legislative Decree 152/2006 and subsequent amendments and Ministerial Decree 161/2012).

In case of selection of quarry material, it shall belong to class A2-A3 of the classification UNI 10006, whose particle size distribution, permeability value and degree of constipation shall be defined in the design and checked as the trench is being refilled by means of Lefranc, Proctor and on site density tests to be run along the path of the pipeline every 250-300 m.

Test results will be analyzed by the works director who may require changes to compaction in case of differences between the characteristics of the compacted material and the characteristics of the surrounding soil.

Finally, the material must be compacted with a manual compactor up to 50 cm above the pipe, subsequently superimposing layers of filling material 30 cm thick each.

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