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Evaluation of Dynamic Soil Properties for Alluvial Plain of Bejaia Using Field Data and Laboratory Tests

Mohamed Khiatine (ORCID) · **Philippe Reiffsteck** · **Ramdane Bahar**

Abstract This paper deals with the evolution of the shear modulus G and the damping D as a function of the shear deformation of the soil, using data from field and laboratory tests. The studied experimental site is located in port area of the city of Bejaia (Algeria), which has already been the subject of numerous geotechnical and geophysical investigation campaigns as part of construction of important industrial projects. In a first step, the geological, tectonic and hydrological contexts of the region are described, as well as the results of the geophysical and geotechnical tests. The result of this first step is the development of a database from field and laboratory tests. In a second step, monotonic and cyclic triaxial tests are performed to identify the dynamic properties on a reconstituted silty clayey sand soil in the laboratory with a characteristic relatively close to the ground in place. In a third step, a numerical analysis is carried out using a FLAC^{2D} code. The nonlinear elastic model proposed by Ramberg–Osgood and limited by the Mohr–Coulomb criterion is used. The comparison between the numerical results and the experimental data shows the applicability of the Ramberg–Osgood model to describe the dynamic behavior of muddy silty sand and muddy sands of Bejaia.

Keywords Shear modulus · Damping · Degradation · Nonlinear model · Dynamic parameters · Geotechnical and geophysical tests

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1 Introduction

The historic seismicity of Algeria shows that the northern part of the country is a highly seismic zone (CRAAG). Over the past four decades, it has experienced several moderate to strong earthquakes. One of the spectacular aspects of the seismic event is the collapse of the structures, relating to the degradation of the shear strength of granular or sandy and silty or clay soils of low relative density. The shear strength decreases or cancels out after a number of shaking

cycles during the earthquake. Under these conditions, the soil behaves like a liquid, which does not resist to shear stresses. For this purpose, knowledge of the cyclic behaviour of soils is essential for major projects in order to understand the seismic response of sites. During the past 30 years, several studies have been conducted to understand the behavior of soils during cyclic loading (Atkison and Stallebrass 1991; Tatsuoka et al. 1997). Research shows that soil behavior is nonlinear and hysteretic characterized by curves giving the evolution of shear modulus $G(\gamma)/G_{max}$ and damping $D(\gamma)$ as a function of shear deformation (Bayat and Ghalandarzadeh 2017; Borel and Reiffsteck 2006; Braja 1983; Kallioglou et al. 2009; Pecker 1984; Seed and Idriss 1970; Shivaprakash and Dinesh 2017; Verruijt 1996; Wang et al. 2018). These curves are generally identified from in situ tests and cyclic laboratory tests performed on representative specimens of the soils. In practice, soil dynamic properties are collected from in situ geophysical tests by measuring the velocities Vs of shear waves, or by laboratory tests, measuring directly the variations of shear modulus and damping versus the deformation.

The Algerian Parasismic Regulation (RPA 2003) requires geophysical survey for sites designed to receive strategic structures, especially in regions of high seismicity to classify these sites based on shear velocity measurements and to evaluate the dynamic properties, in particular the dynamic shear modulus on very low deformation soils over the first 30 m of depth. Failing this, this classification may be made on the basis of the results of in situ tests such as the pressuremeter test (PMT), the cone penetration test (CPT) and standard penetration test (SPT). The initial modulus of the soil in place is an important factor in the design of certain structures subjected to small deformations. Nevertheless, one of the obstacles found in the design phase of Algerian projects, the shear modulus degradation and damping curves are not available for Algerian soils.

This paper discusses an experimental and numerical analysis to evaluate the degradation curve of the shear modulus $G(\gamma)/G_{max}$ and the evolution of damping $D(\gamma)$ according to the distortion of the experimental site located in the harbor area of Bejaia, Algeria. This paper is divided into two main parts. The first part consists of an experimental study. This involves analyzing all the experimental data relating to industrial projects in the port area of the city of Bejaia.

This part also includes performing monotonic and cyclic triaxial tests and resonant column tests to evaluate the degradation shear modulus and the damping curves of reconstituted silty clayey sand soils relatively close to the soil encountered in Bejaia site. In a second part, a numerical identification of these curves is performed using the FLAC^{2D} code. The nonlinear elastic model proposed by Ramberg–Osgood and limited by the Mohr–Coulomb criterion is used (Bagagli et al. 2010).The empirical curves given by Seed and Idriss (Seed and Idriss 1970) for granular materials and Vucetic and Dobry (Vucetic and Dobry 1991) for fine-grained soils are used to calibrate the model parameters.

2 Experimental Site: Harbor Area of Bejaïa

2.1 Location and Morphology

Bejaïa is an city located on the sea shore of the Mediterranean Sea, about 180 km east of Algiers the capital city of Algeria. It is a very populated city approximately 180,000 inhabitants, it is also known for its big industrial zone and its oil and commercial port. The city of Bejaia has clung to the slopes of Djebel Gouraya, and then extended southward into the plain. The harbor area was developed at the mouth of oued Soummam and land reclaimed from the sea (Fig. 1).

2.2 Geological Context

The regional geology materializes the Bejaia Plain in the post-table syncline basins of Tell (Roth 1950), the depression formed between Mount Gouraya in the north and the port area in the south was filled with fine alluvial deposits of the Oueds Soummam and Seghir and interspersed in transgressive marine deposits. Land located in the alluvial plain of Bejaia, particularly in the port area of the city, have unfavourable geomechanical characteristics. The particularity of the site is its low bearing capacity, high compressibility and the potential risk of liquefaction and flooding (Bahar et al. 2011; Delft-Laboratory 1978; Khiatine 2011; Sadaoui 2006). The study site is located at the river mouth of oued Soummam and close to the port of Bejaia. From a stratigraphic point of view, all deposits are of Quaternary age. It follows from the geological
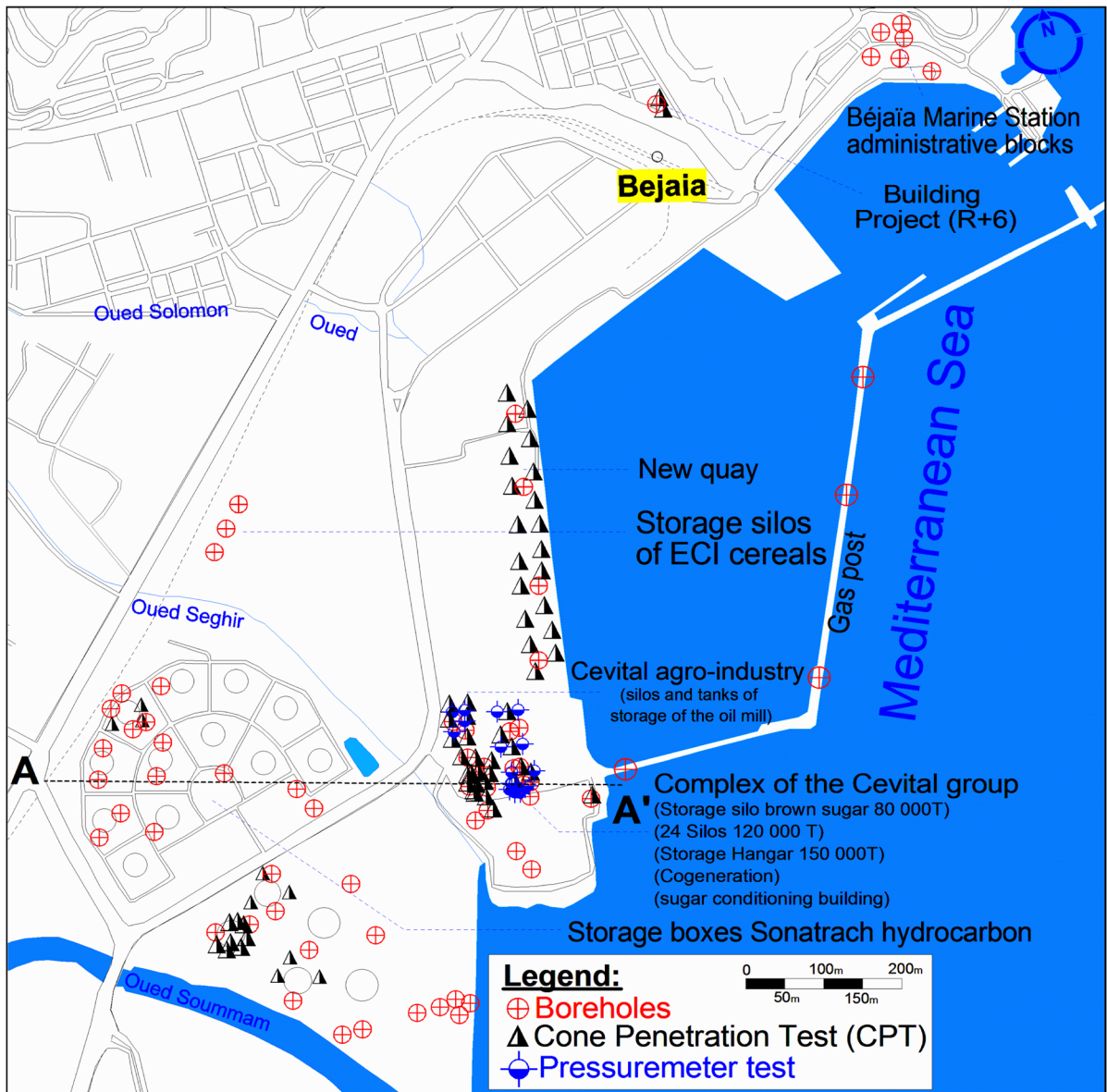
Fig. 1 View of the experimental site in the Bejaia harbor area

history that the port area extending the alluvial flood plain consists mainly of loamy and clay loamy sandy nature, impregnated with a silty matrix deposited on a rather deep substratum encountered at about 40–50 m of marl-limestone cretaceous nature (Fig. 2).

2.3 Tectonics and Seismicity

Northern Algeria is located on the border of two tectonic plates: Eurasian plate and African plate with a shortening speed between the two plates of the order of 5–6 mm/year. Induced earthquakes are generally due to a reverse failure mechanism (AFPS 2003). At the local level, the examination of the historical seismicity of the Soummam and its surroundings since 1949 to date shows a moderate intensity compared to the destructive earthquakes of El Asnam (1980) and Boumerdes (2003). In the past, several earthquakes had shaken the region in the wake of several earthquakes, causing human losses and major
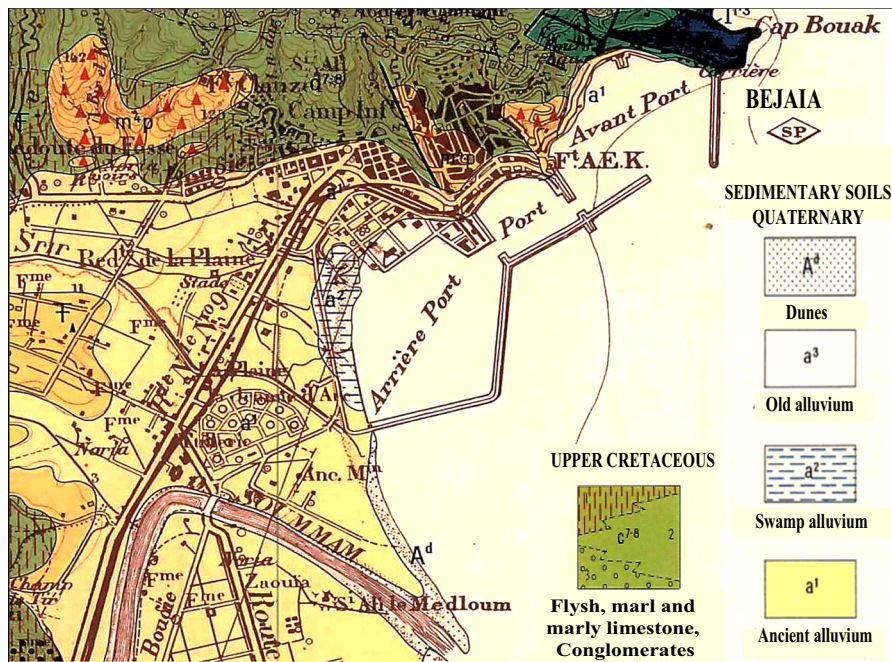
Fig. 2 Extracted of geological map n°26 of Bejaia at scale of 1/50 000 (Sheet-n°26 1960)

disorders on buildings. The earthquake in Bejaia (1960) had caused major damage, with 264 victims, 112 injured and 1000 houses destroyed. The recorded magnitude according to CRAAG is 5.6 on the Richter open scale (CRAAG). The golf of Bejaia was also epicenter of an earthquake on February 25, 1968. The earthquake of Beni Ourtillane (November 2000) and the last one of Babors (22/05/2006) of magnitude of 5.8 on the open scale of Richter according to CRAAG. Bejaia region is classified in zone IIa, medium seismicity according to the Algerian seismic rules RPA (RPA 2003).

2.4 Hydrogeological and Hydro-Climatology Context

The city of Bejaia has a Mediterranean climate characterized by a warm, dry summer and a rainy, wet winter. Alluvial fill formations are represented by alluvial terraces are highly permeable, considered to be the main hydrogeological formations. Identified in the Soummam valley, they constitute a good aquifer because of their large thickness in coarse elements. Furthermore, the Jurassic limestone formations of Gouraya contain some sources of relatively large water resurgences such as the source of Toudja (flow greater than 10 l/s), thus testifying to the great potentiality of groundwater.

2.5 Geotechnical Investigations of the Bejaia Harbor Area

As part of the implementation of major industrial projects in the harbor area of Bejaia city of namely agro-industrial (silos and oil mill storage tanks), the refineries (storage silo of 80,000 T, 24 storage silos of 120,000 T, brown sugar hangar stocking 150,000 T, cogeneration, packaging building), Cereal storage silos, hydrocarbon cylindrical steel storage tanks, new quay, gas post, maritime station, building in R + 6. The analysis of soil geotechnical reports has allowed us to synthesize the soil survey program for the entire area as follows: approximately seventy (70) boreholes drilled from 5 to 51 m deep, one hundred eighty (180) standard penetration tests SPT, about eighty (80) static cone penetration tests CPT, eight (08) surveys of down hole tests from 1 to 42 m deep or about 217 measurements, eighteen (18) pressuremeter sounding from 4 to 30 m deep and laboratory tests (physical identification tests, Atterberg limits, particle size analysis, mechanical tests, and chemical analyses).

The collection of data from the numerous geotechnical studies carried out in the harbor area of the Bejaïa plain (Figs. 1 and 3) resulted in the elaboration of a database in situ geotechnical test results (pressuremeter, CPT and SPT tests) and geophysical tests (down hole) as well as laboratory tests (physical, mechanical and chemical analyses).

In geotechnical terms, most of the sites were covered by investigation tests to evaluate soil resistance and their degree of constructibility. It turns out that those superficial layers of marshy alluvial nature, predominantly sandy-silty to clayey, impregnated with a muddy matrix and sometimes heterogeneous (Fig. 3), have not yet reached a sufficient degree of consolidation, therefore, their bearing capacity is low and their compressibility is high. These layers of alluvial nature deposited on a marl-limestone bedrock about 25–45 m deep. It is also important to note that several cases of structures built on superficial foundations have experienced some distresses and even failure, such as the generalized collapse of petroleum reservoirs and the settlement of the bridge crossing oued Seghir (Sonatrach, 1991, 2004).

2.5.1 In-Situ Tests

The site has been the subject of several geotechnical investigation campaigns since 1970 to 2012. Examination of the lithological sections of the boreholes

confirm the stratigraphy and give the ground model shown on the Fig. 3. The alluvial deposits are represented in mass by muddy silty fine sands brownish-brown on the surface and bluish-grey in depth, on a thickness ranging from 3.00 to 42.00 m, in which layers of highly plastic sandy vases of bluish-grey colour are interspersed (Fig. 4), especially between 6 and 12 m deep. These deposits surmount a layer of beige to yellowish gravelly sandy marl with a passage of pebbles of varying thickness between 39 and 45 m. The whole complex is based on a compact marly bedrock of greyish colour (Fig. 5).

The mechanical parameters deduced from the penetration (CPT, SPT) and expansion tests (PMT) show the low soil resistance (Figs. 6 and 7). For this effect, the results of the static penetration tests indicate peak cone resistances (q_c) of less than 1.0 MPa on certain layers, particularly between 5 and 13 m deep. This indicates the intercalation of low-strength layers of soft clays or vases. Examination of the different profiles also shows the fairly pronounced sawtooth appearance of the peak resistances with depth in all the performed tests.

The geophysical investigation by Down Hole logging was carried out in accordance with the procedure prescribed by ASTM D4428/D4428M. This test is used to determine the shear velocities for a section of terrain located in the proximity of the borehole, and for a vertical wave propagation direction
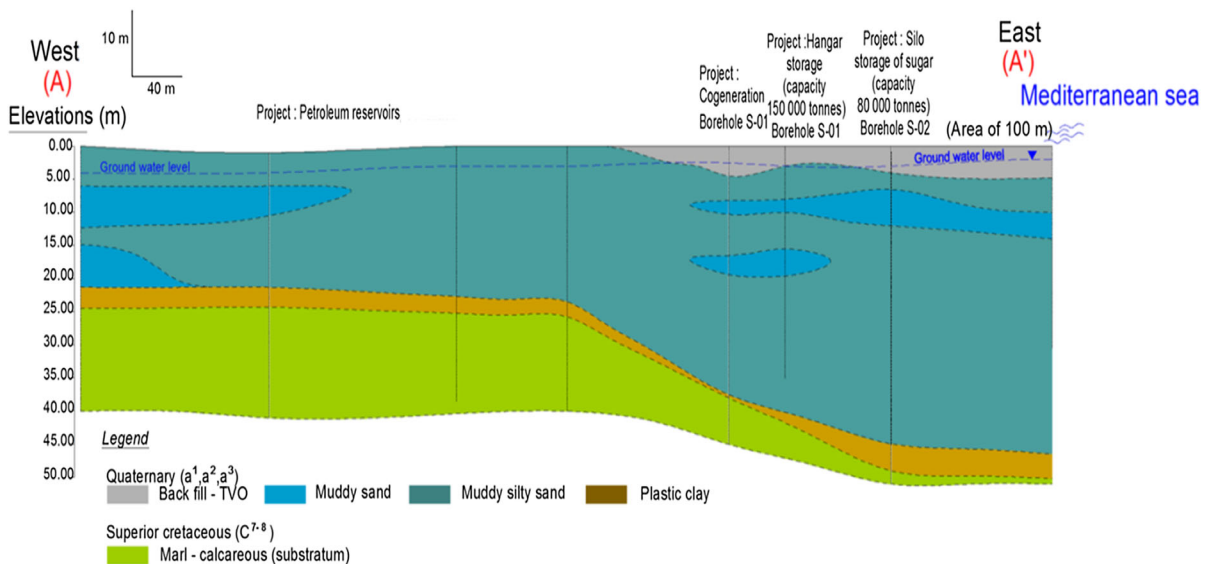

Fig. 3 Geological section A-A′ of the Bejaia harbor area
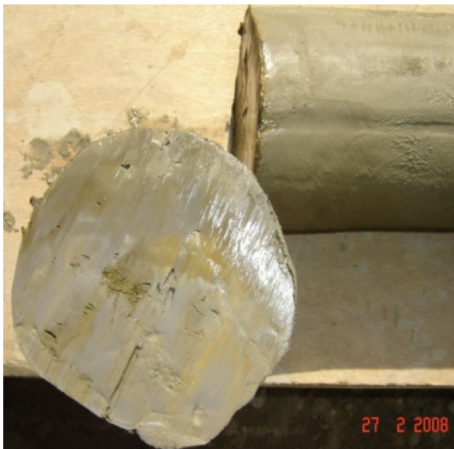
Fig. 4 Muddy fine sands and muddy

Fig. 5 Gray Marl

using the Down Hole method. This method consists of measuring the velocities of compression (V_p) and shear (V_s) waves of a geological formation between the ground surface and the position of the sensor in the borehole. The Down Hole test is carried out in the core borehole with a PVC core tube 80 mm inside diameter and cemented to ensure a good ground-to-tube coupling. Eight (08) Down Hole geophysical prospecting drill holes from 20 to 42 m depth were drilled on the plate of Cevital group complex between 23 and 30/03/2008. A total of 217 seismic velocity measurements were performed. The measurements are given below (Fig. 8).

2.5.2 Laboratory Tests

Intact samples taken from the various boreholes and depths were subjected to physical and mechanical tests in the laboratory. The results of these tests are shown in Figs. 9 and 10 (Fig. 11).

The Quaternary age superficial geological formations of the site, composed of alternating soft clay and muddy sands, are characterized by low to medium densities. The Atterberg limits indicate a plastic to medium plastic soil. The parameters obtained from the oedometric tests reflect a soil that is moderately compressible to highly compressible. The shear strength obtained in these muddy sands is quite variable. This variability can be explained by the presence of relatively compact layers that influence the apparent shear strength during successive phases of testing. However, the calcareous marl bedrock is characterized by a medium density to dense soil. The Atterberg limits indicate medium plastic soil (Fig. 12). The results of the oedometric tests show that the soil is normally consolidated, moderately compressible.

The granulometric analysis carried out on the samples recovered in the SPT corer and in the drill holes drilled over the first 36 metres of depth revealed generally a grevelly soil with a predominant medium-to-fine sand fraction, with the presence of a clay fraction on certain depths of less than 24%. According to the values of the uniformity coefficient, the obtained curvature coefficient and according to the LCPC
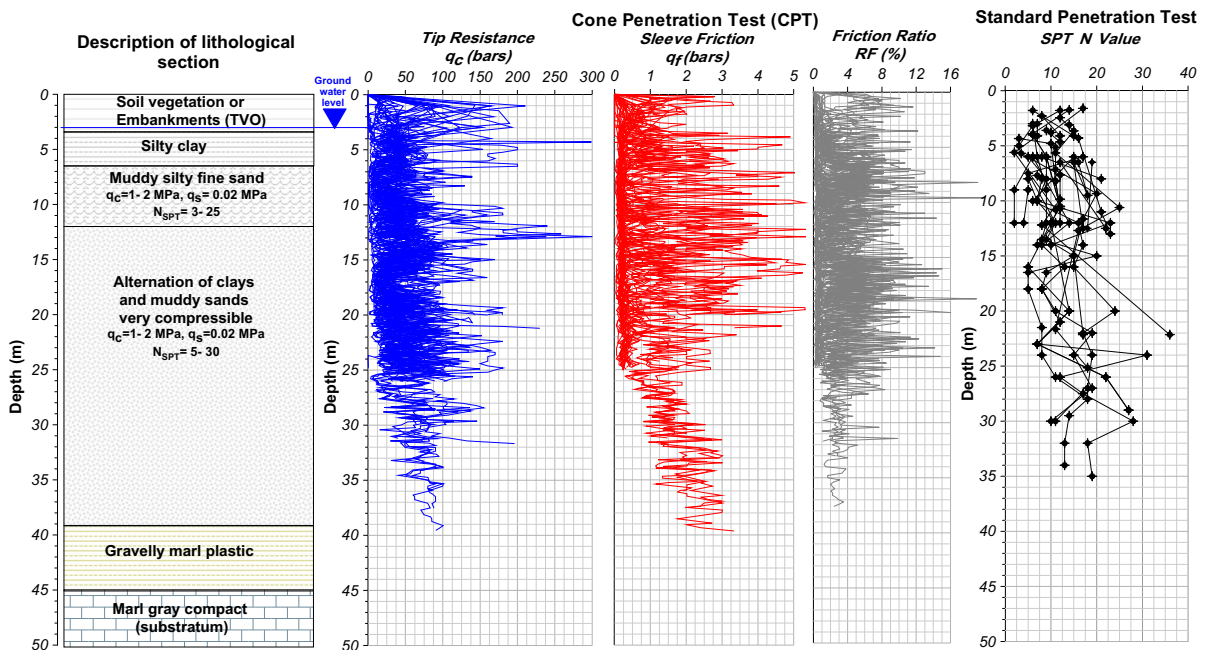
Fig. 6 Penetrometric parameters measured of Bejaia harbor area by in situ tests (CPT and SPT)
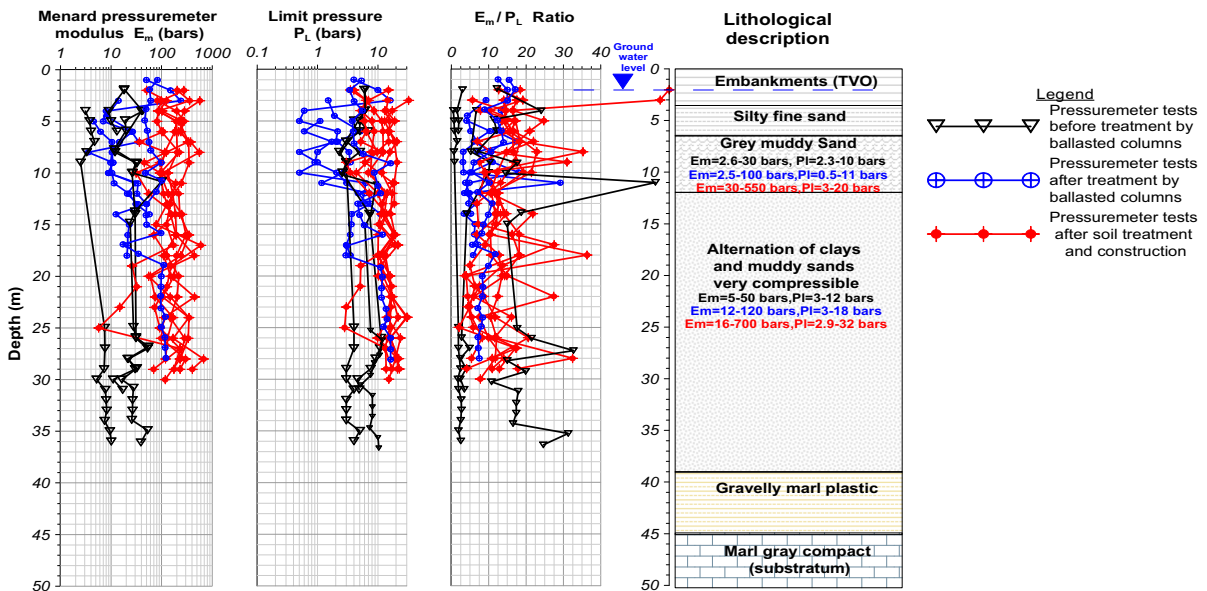

Fig. 7 Pressuremeter parameters results measured of Bejaia harbor area

classification the soil analyzed is classified in the category of clean sands well graduated.

Robertson and Campanella (1983) presented the effect of sand compressibility on the relationship between the cone tip resistance (q_c) and the relative density (D_r) using three different qualitative compressibility categories (highly compressible sands, normal compressible sands and low compressible sands), as shown in Fig. 13. According to this chart, the relative densities of the in situ soils are between $D_r = 20\%$ and $D_r = 60\%$ according to the Schmertmann classification.
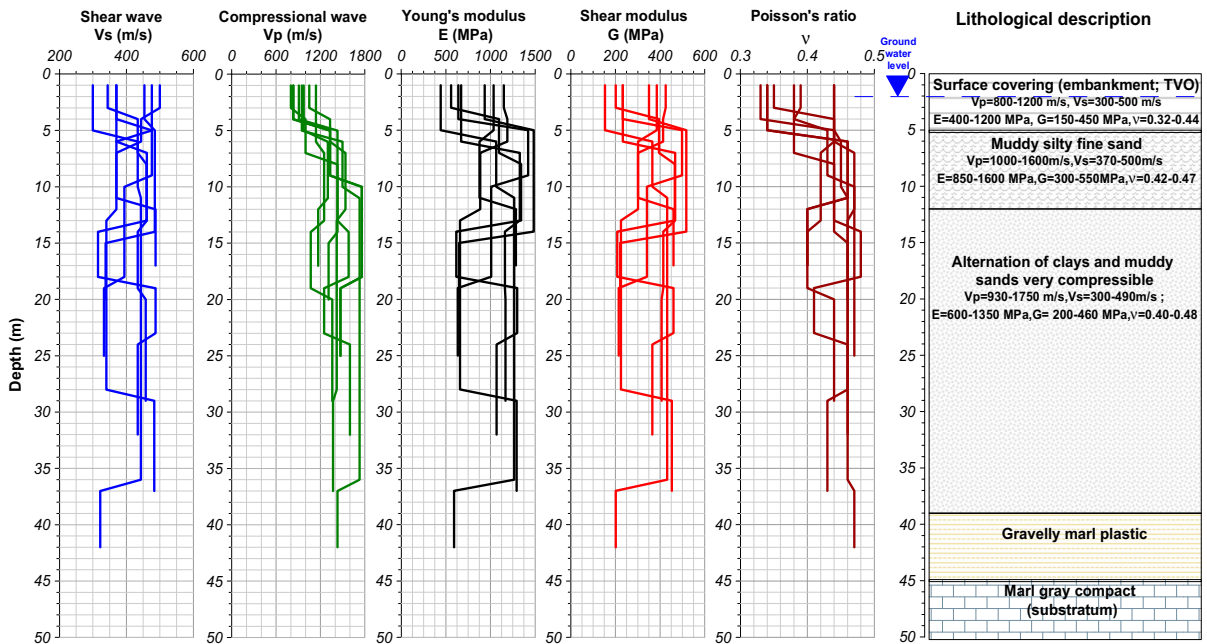
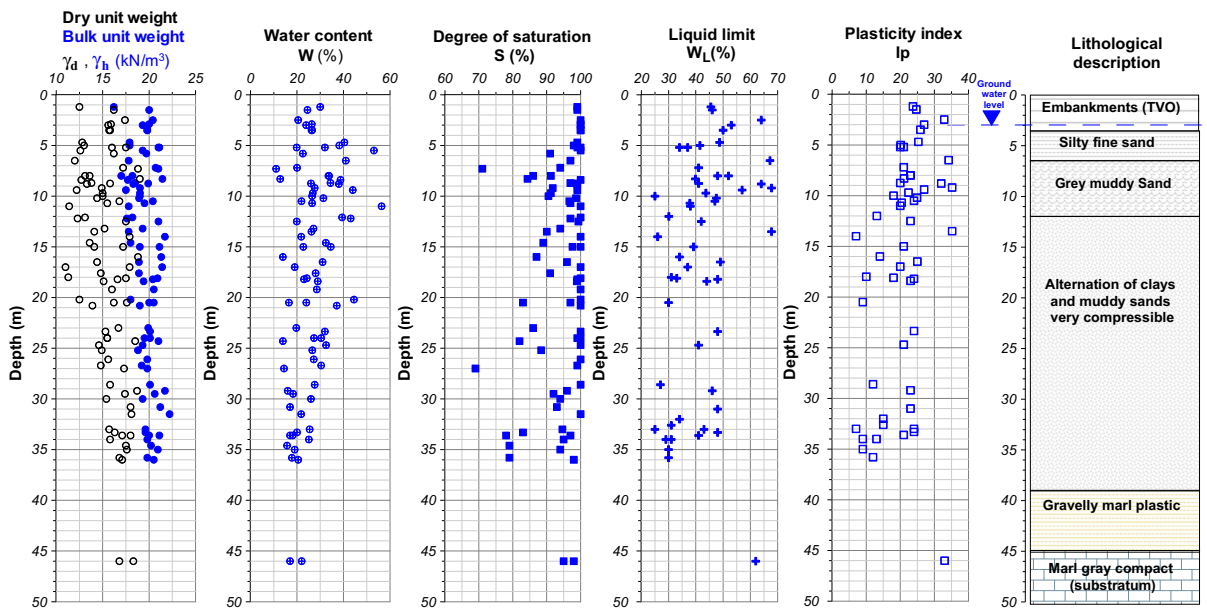**Fig. 8** Down hole geophysical test record results in the harbor area of Bejaia

Fig. 9 Physical properties of soils

SPT N-value gives a qualitative guide to the in situ engineering properties and provides an indication of the relative density and friction angle of the soil as proposed by Meyerhof (1956) (see Table 1).

According to this Table 1, the alluvial formation forming part of the Bejaia plain's over the first 30 metres of depth characterized by a N_{SPT} blow count of
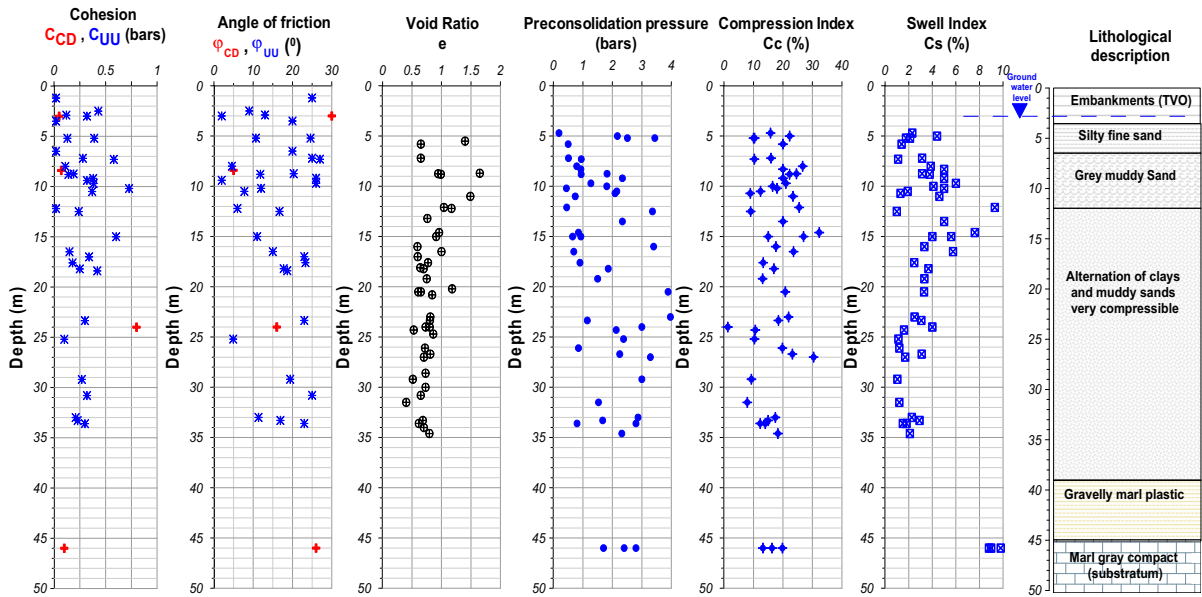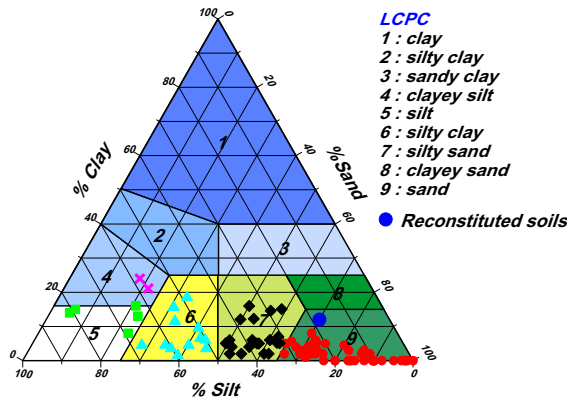
Fig. 10 Mechanical properties of soils

Fig. 11 Textural soil classification in a ternary diagram with LCPC denomination

Fig. 12 Casagrande plasticity chart existing and reconstituted soils (USCS) (Casagrande 1948)

3 and 30 correspond to relative densities between 20 and 60%.

2.5.3 Discussion of the Results

Most of the land located in the alluvial plain of Bejaia presents geotechnical constraints in terms of low bearing capacity, high compressibility, susceptibility to liquefaction and flooding for land reclaimed from the seaside, which makes use of the technique of soil improvement by stone columns in order to limit these major risks for several industrial projects located on
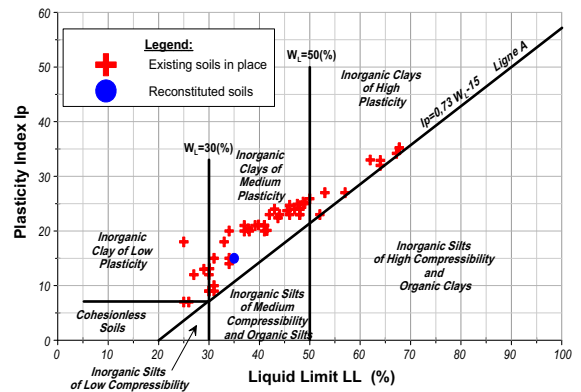
the alluvial plain (Bahar et al. 2011; Khiatine 2011; Sadaoui 2006; Sadaoui and Bahar 2017). It is noted that the results of the penetrometer and pressuremeter tests are consistent with those of the laboratory and confirm that the alluvial formation composed of alternating layers of loose sand, muddy sands and silt is in an under-consolidated to normally consolidated state. According to the results of the SPT and CPT tests, the alluvial formation belonging to the filling of the Bejaia plain over the first 30 metres of depth characterized by relative densities between 20 and 60%.
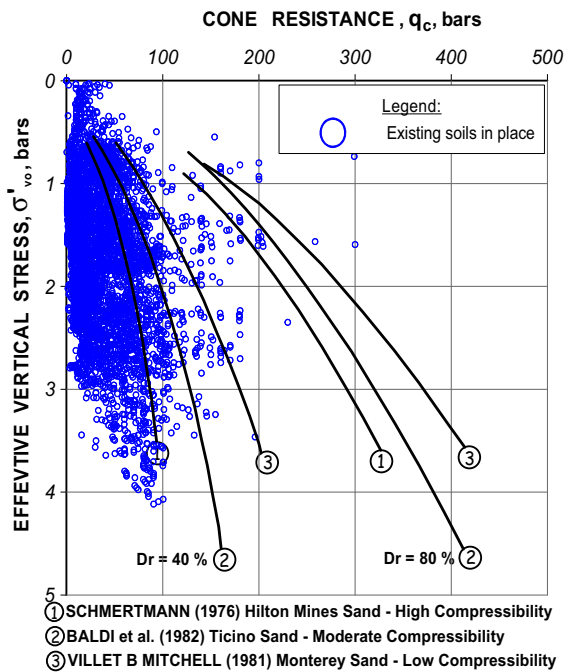
Fig. 13 Relative density Results of soils of Bejaia harbor area based on CPT tests (Robertson and Campanella 1983)

3 Experimental Study of Monotonic and Cyclic Behavior

An experimental parametric study has carried out on a soil reconstituted with the triaxial apparatus. The study has three main parts. This part concerns the experimental study carried out on a soil reconstituted with the triaxial apparatus. The study has two main parts. The first stage was devoted to the search for a soil with a characteristic relatively close to that of the Bejaia experimental site (Atkison and Stallebrass 1991; Tatsuoka et al. 1997). This step of creation consisted in making several mixtures in order to get as close as possible to the existing soil on the first 30 meters of depth, several mixtures were made in the IFSTTAR laboratory (Figs. 14 and 15). After several limits tests

and granulometric analyses, the mixture composed of 70% Fontainebleau sand, 18% artificial silt and 12% bentonite is as close as possible to the soil in place with respect to plasticity and granulometry (Figs. 12 and 16).

To get an idea of the compactness of the reconstituted soil in which it is naturally present, tests were carried out to determine maximum and minimum density of soil γ_{dmin} and γ_{dmax} corresponding to soil ratio. The results of the tests carried out on the mixture are given in Table 1.

In the second part, the behaviour of reconstituted soil under monotonous loading, conventional drained tests are presented. The interest of these tests lies in the fact that they lead to a reduction of the effective stresses and that they allow to test the influence of the stress path on the fracture in terms of the appearance of loss of stability as well as in terms of achieving a stable state of deformation (Table 2).

Several experimental studies have been carried out to analyse and understand the fundamental phenomena that govern soil behaviour. This section examines the main behavioural characteristics of saturated soils, studied mainly in the triaxial apparatus. It is the most widely used laboratory test in soil mechanics, it allows the determination of the shear strength of the soil under different drainage conditions. The test consists of placing a cylindrical soil specimen under a uniform stress field defined by hydrostatic pressure in the horizontal plane and vertical stress.

The experimental device used to carry out the triaxial tests presented in Fig. 17 is that of IFSTTAR in Marne-la-Vallee. The triaxial cell GDS brand 2 Hz, can contain specimens with diameters of 50 mm, and withstand fluid pressures up to 2 MPa. The device also includes three GDS pressure/volume controllers and a microcomputer equipped with software for test control and data acquisition. Otherwise, the triaxial apparatus can perform drained and non-drained compression and extension, monotone and cyclic, deformation or controlled stress tests.

Table 1 Characteristic of the reference soil (reconstituted soil)

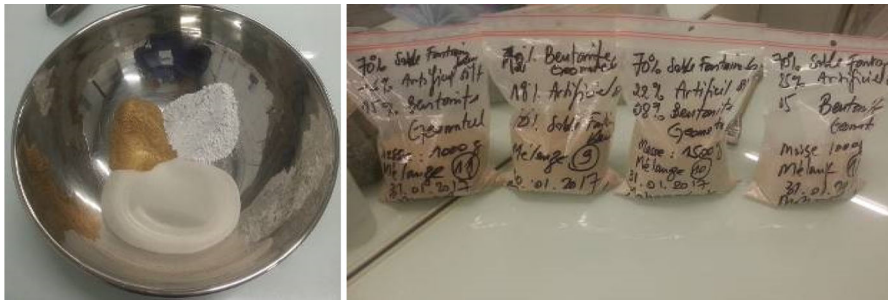| Reconstituted soil	FC (%)	W_L	Wp	Ip	D_{50} (mm)	e_{min}	e_{max}	γ_s (kN/m³)	γ_{dmin} (kN/m³)	γ_{dmax} (kN/m³)
70% Fontainebleu sand 18% artificial silt + 12% bentonite	30	35	20	15	0.170	0.213	0.745	26.5	15.20	21.85

Fig. 14 Mixture of Fontainebleau sand + artificial silt + bentonite

Fig. 15 Liquidity limits (WL), plasticity (Ip) and particle size analysis of the mixture
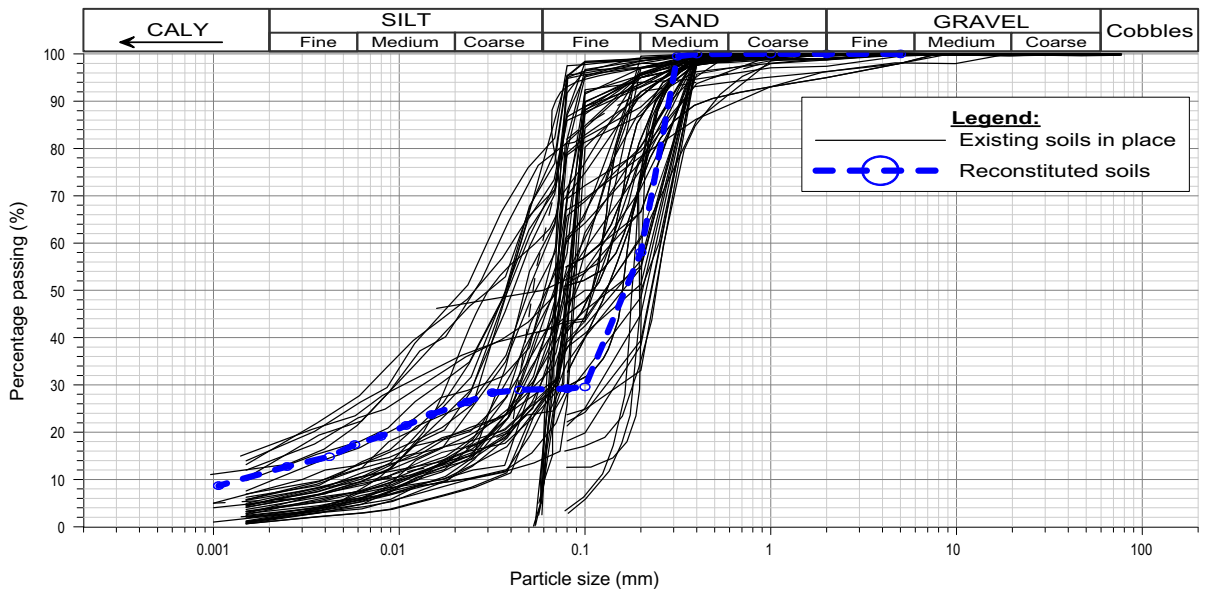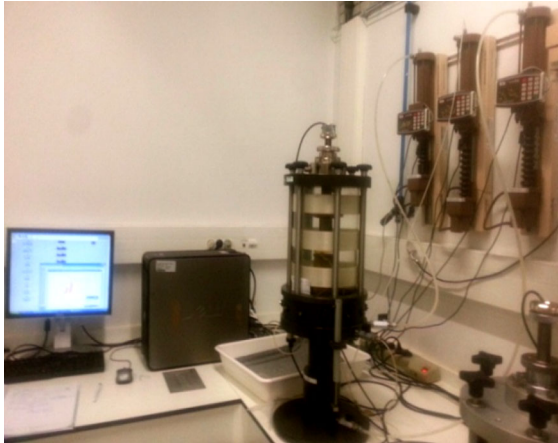

Fig. 16 Grain size distribution curves of soils of Bejaia harbor area and reconstituted soils in laboratory

Table 2 Model parameters

Layers of soil	Thickness (m)	ρ (kg/m³)	C' (kN/m²)	φ' (°)	r	α	V_s (m/s)	G_{max} (MPa)	K_0 (MPa)	G_0 (MPa)
Muddy silty sand and muddy sand	0.00–30.0	1620	26	10	2.2	1.4	445	320	1825	173

Fig. 17 Experimental device used that of IFSTTAR

Fig. 18 Sample preparation with mold

The soil specimens are produced using a cylindrical metal mould 50 mm in diameter and 100 mm high (Fig. 18), on which the mixture of the three materials obtained by mixing is poured to ensure a good homogeneity of the reconstituted soil. Samples were placed in the mould using the Ladd method (Ladd 1978), modified by Chan (1985) which consists in dividing the total quantity of the material into several successive layers (five layers). The soil mass to be installed is evaluated according to the desired density (the initial volume of the sample is known), the density state of the sample being defined by the relative density:

$$D_r(\%) = (e_{max} - e) / (e_{max} - e_{min}) \times 100 \qquad (1)$$

Depending on the relative density of the existing soil in place, the specimens are prepared with various densities ranging from D_r = 20, 40 and 60%. Once the specimen is set up at the desired density, the specimen is transferred to a cold room to preserve the physical characteristics of the sample. Three series of tests of the consolidated drained (CD) were carried out as follows.

Reconstituted samples were made for three values of the consolidation stress (50,100 and 200 kPa). This choice is justified by the fact that highly compressible soft muddy sand deposits of the harbor city of Bejaia are frequently encountered from the surface to a depth of 30 m, which corresponds to an effective vertical stress of about 200 kPa; Triaxial tests of the CD consolidated drainage type were carried out at a speed of 0.007 mm/minute according to the French standard (AFNOR)(NF P 94-074) in order to obtain a precise understanding of the volume changes after dissipation of the interstitial pressure and perfectly reproduces the geostatic conditions of the in situ soil and ensures a more satisfactory control of the soil behaviour.

Before carrying out the consolidation, the saturation state of the specimens was checked by determining the Skempton B coefficient. For all the tests carried out, it was obtained closer to or greater than 0.90, which makes it possible to verify the good saturation of the specimens.

During the consolidation phase, the degree of consolidation, volume deformation and corrected volume of the sample are measured over time. In the shear phase, the stress deviator, axial deformation and volume variation of the specimen are recorded.
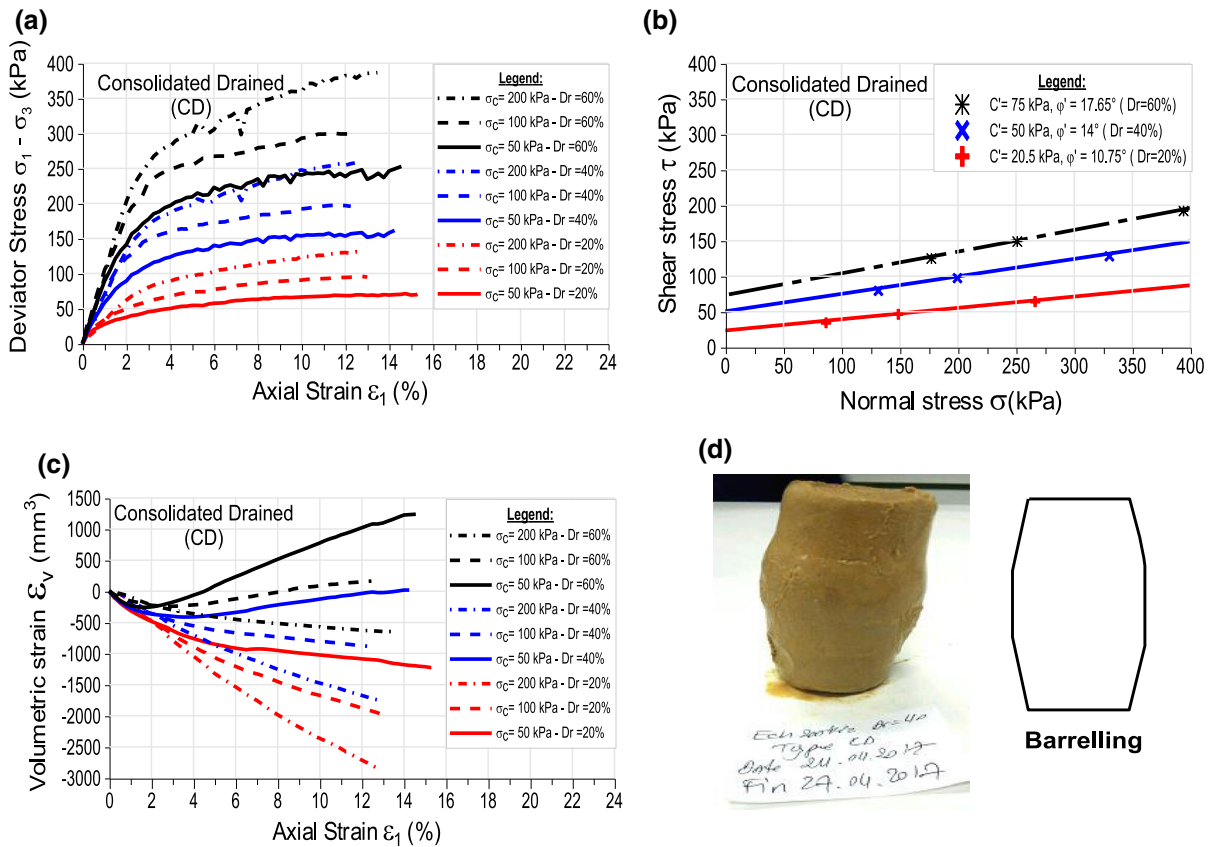
Fig. 19 Drained response of reconstituted soil (confinement = 50,100 and 200 kPa for different densities Dr = 20, 40 and 60%)

Figure 19 shows the results of triaxial tests performed on reconstituted soil under drained conditions.

3.1 Discussions and Comparisons of Results

The comparison between the physical and mechanical properties measured from tests performed on reconstituted and intact samples shows a relatively similar characteristics, and also close to those obtained by other authors (Sadaoui and Bahar 2017). Figure 19a shows the results of monotonic drained type triaxial tests carried out on reconstituted soil for different relative densities and distinct confining pressure. It is noted that the relative density and the confinement significantly affect the deviator and the volumetric deformation variations. The increase of the relative density between 20 and 60% induces an increase of the initial stiffness of the soil and the resistance of the soil (maximum deviator). As regards the volume deformation (Fig. 19b), it is noted that the 60% relative

density samples with fines fraction (Fc < 30%) exhibit a phase of contractance following a dilatancy phase. For samples with a relative density of 20 and 40%, only a contract phase is observed. Note that the difference between these curves decreases with the decrease of the relative density and the confining pressure, reflecting the disappearance of the dilatancy phase and the appearance only of the contractance phase from relative density 60% with a confining pressure of 50 kPa (Fig. 20).

4 Experimental Evaluation of Dynamic Properties

In geotechnical engineering, the design of structures sometimes requires a very precise knowledge of displacements and soil deformations. In many practical cases, the level of deformation involved does not exceed 10^{-3}. Several studies and research studies have favoured the use of shear modulus degradation and

Fig. 20 Range of application of test instruments for the determination of deformation modulus (Reiffsteck 2002)
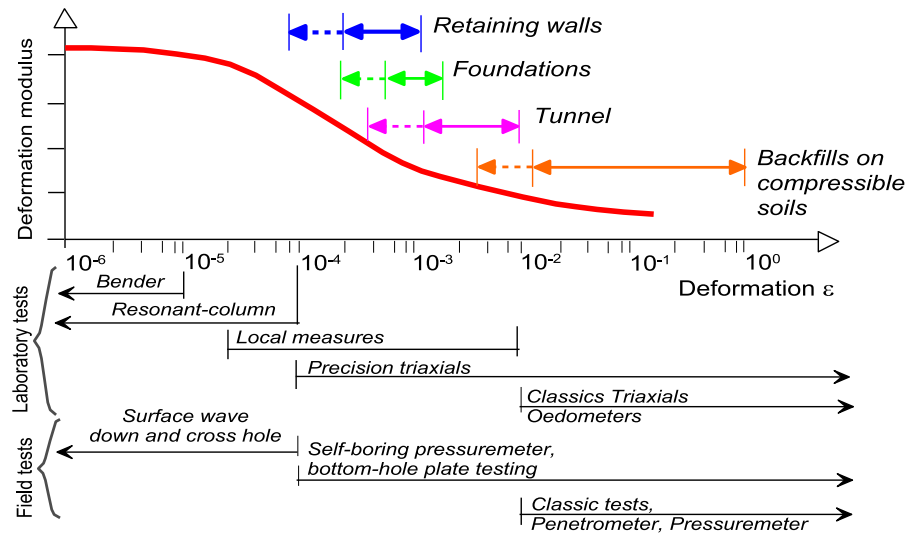
damping according to the level of deformation, especially after having been identified as key parameters for geotechnical problems of dynamic and static aspects. The soil's dynamic properties are then accessible through a number of laboratory test techniques (Atkison and Stallebrass 1991) and in situ tests (Tani 1994; Tatsuoka et al. 1997), each test having a field of application related to the deformations it generates to solicit soil (Reiffsteck 2002).

In order to determine the shear modulus on a wide range of strain, we have combined the resonant column tests that allow the determination of the desired parameters for very small to small deformations ($10^{-6}<\gamma < 10^{-4}$), cyclic triaxial tests for medium to large deformations ($\gamma > 10^{-4}$) and classical pressuremeter tests for large deformations ($\gamma > 10^{-3}$). The resonant column and triaxial tests were performed on saturated samples.

4.1 Pressuremeter Tests

The results of the Menard pressuremeter tests are used to estimate the shear modulus values of the soil in place. In this type of test, the shear modulus G is measured directly from the soil. It is expressed by the following expression:

$$G = V \cdot \frac{\Delta P}{\Delta V} \qquad (2)$$

where ΔP, is the pressure variation applied to the cavity border; ΔV, is the variation of the cavity volume corresponding to ΔP; V, is the volume of the initial cavity.

However, this relationship supposes a linear isotropic elastic behaviour of the soil in which the test is performed. Of course, that is not the case. This type of relationship is, however, still applied for the routine operation of pressuremeter tests. In this case, an average soil shear modulus is calculated on a range (ΔP; ΔV) of the curve considered to correspond to pseudo-elastic soil behaviour.

$$G = V_m \cdot \frac{\Delta P}{\Delta V} \qquad (3)$$

With Vm = Vo + $\Delta V/2$: is the mean volume of the cavity corresponding to ΔP, G: is the Menard pressuremeter modulus.

In this type of test, the environment soil is tested globally and only the deformation of the soil at the edge of the cavity is measured. It is directly related to the volumetric deformation of the cavity by the following expression:

$$\varepsilon_\theta = -\varepsilon_r = \frac{\Delta R}{R_0} = \sqrt{1 + \frac{\Delta V}{V}} - 1 \qquad (4)$$

where ΔR: is the variation of the cavity radius, R_0: is the initial radius of the cavity, V: is the volume of the cavity.

The deformation of the soil at the edge of the pressuremeter cavity is the maximum deformation that

develops in the massif. This is the one that has been associated with the different pressuremeter modulus measured (module of the Menard type, cyclic modulus). Shear modulus from the pressuremeter tests data are presented in Figs. 21 and 22. Figure 15 show the comparison between the shear modulus Menard and the maximum shear modulus calculated from the segments at the pseudo-elastic soil behaviour.

4.2 Cyclic Tests on the Triaxial Apparatus

The most common laboratory cyclic tests are the triaxial apparatus test, straight-line shear test and hollow cylinder test. Therefore, the cyclic triaxial test system was used to characterize the dynamic properties of reconstituted soil samples. This test device typically consists of a cell-supporting press, a multi-axial control system to control the axial loading, containment stress and back pressure used in the containment phase of the sample. This system applies cyclic loads and automatically records experimental data. Cyclic axial loading is applied in terms of force, deformation or vertical displacement. For the classical aspects, the execution of the test refers to the standard of the triaxial test with monotone load. For cyclic reference is made to the American Standard ASTM D 3999-91(ASTM 1996).

The second part examines cyclical behaviour, which essentially takes the form of two phenomena: cyclical collapse and cyclical mobility. A program of cyclic triaxial tests was carried out on the reconstituted soil with various densities ranged from Dr = 20, 40 and 60% under different confinement (50 kPa, 100 kPa and 200 kPa) for the main purpose of establishing cyclic shear curves for this reconstituted soil (Fig. 23). The sample preparation procedure is the same as for monotonous loads. On the other hand, the type of stress requires the specimen head to be

Fig. 21 Evolution of shear modulus G with cavity deformation (site harbor of Bejaia)
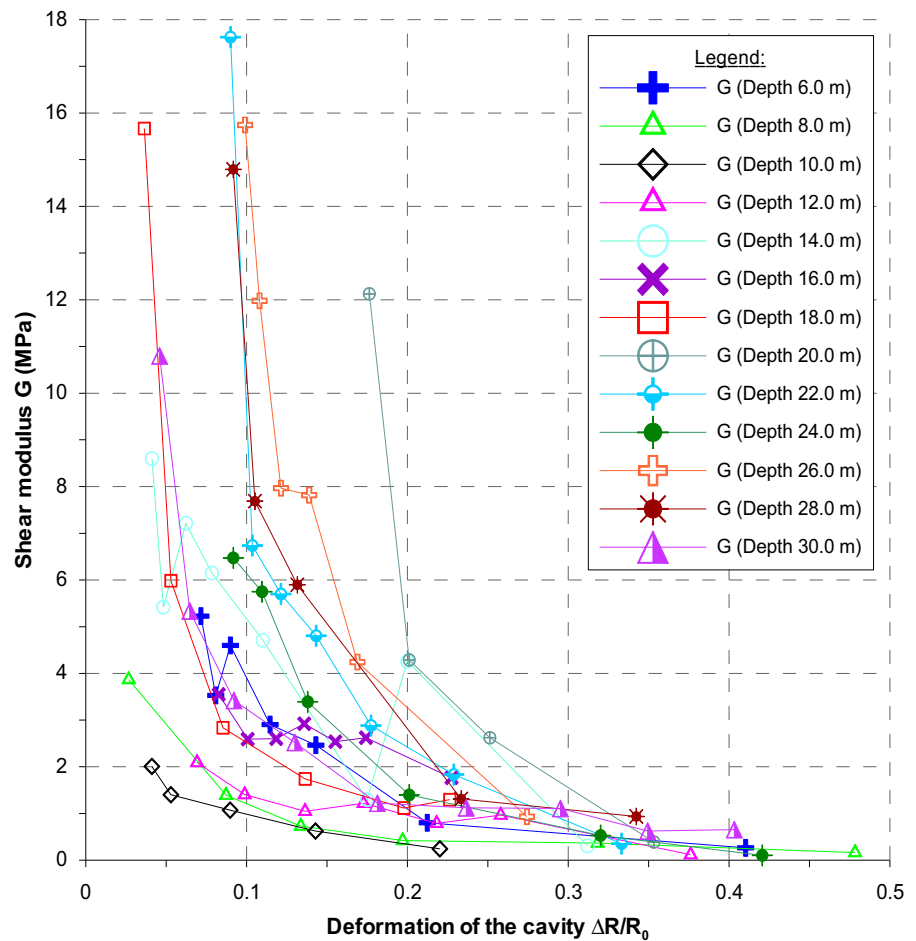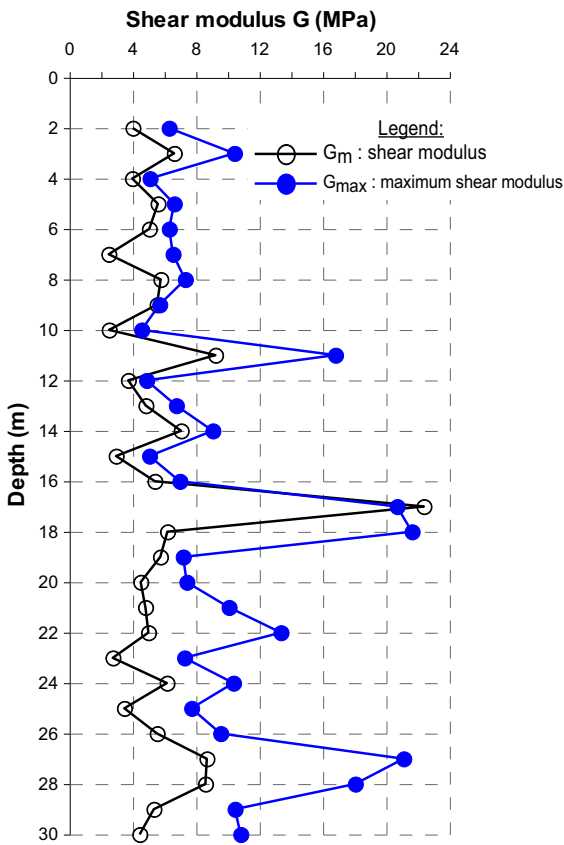
Shear modulus G (MPa)

Fig. 22 Shear modulus and the maximum shear modulus from pressuremeter tests data

attached to the piston. The specimen size is the same as for monotonous tests (50 mm diameter; 100 mm height). The cyclic tests are carried out in no-drained conditions at a frequency of 0.5 hertz. It is also possible to impose different signal forms (sinusoidal, triangular or square). In our case the sinusoidal signal is used.

Undrained cyclic tests were carried out on reconstituted soil relatively close to that of the soil in place for three relative densities Dr = 20, 40 and 60% with different confinement pressures (50, 100 and 200 kPa). For each density, we have varied the amplitude of the load in order to construct the degradation curve for the shear modulus of the reconstituted soil. The results of all these tests are summarized in Fig. 24.

4.3 Resonant-Column Apparatus

The resonant column test is the most commonly used laboratory test for measuring the low-strain properties of soils and particularly the dynamic properties of soils. In third part the resonant column test is used of reconstituted soil. The basic principle of the resonant column test is to vibrate a cylindrical soil sample in a fundamental mode of vibration, in torsion or flexure (Figs. 25 and 26). Once the fundamental mode is established, measurements of resonant frequency and
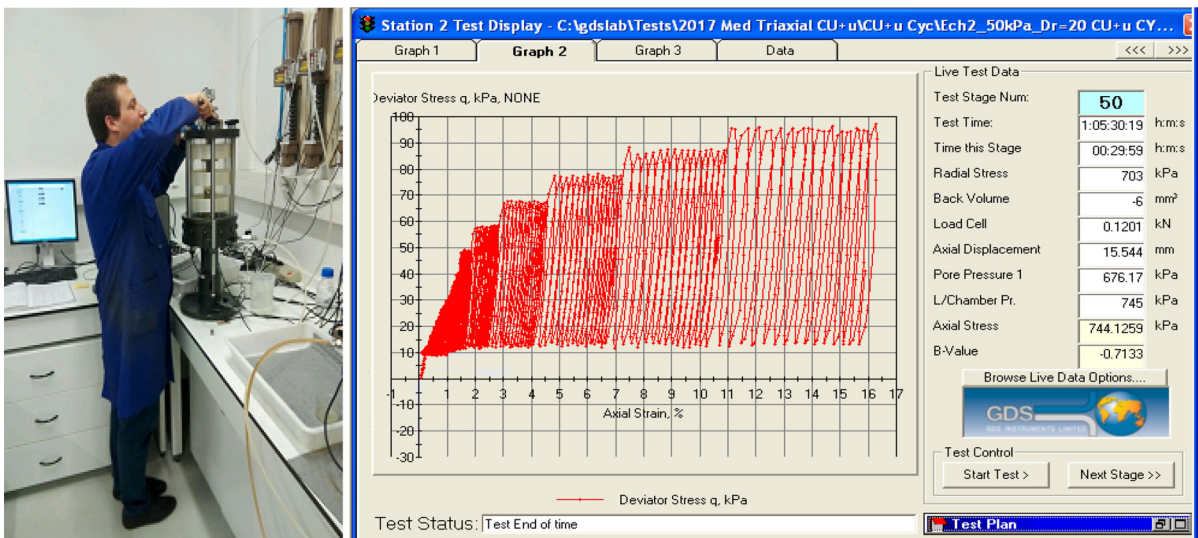

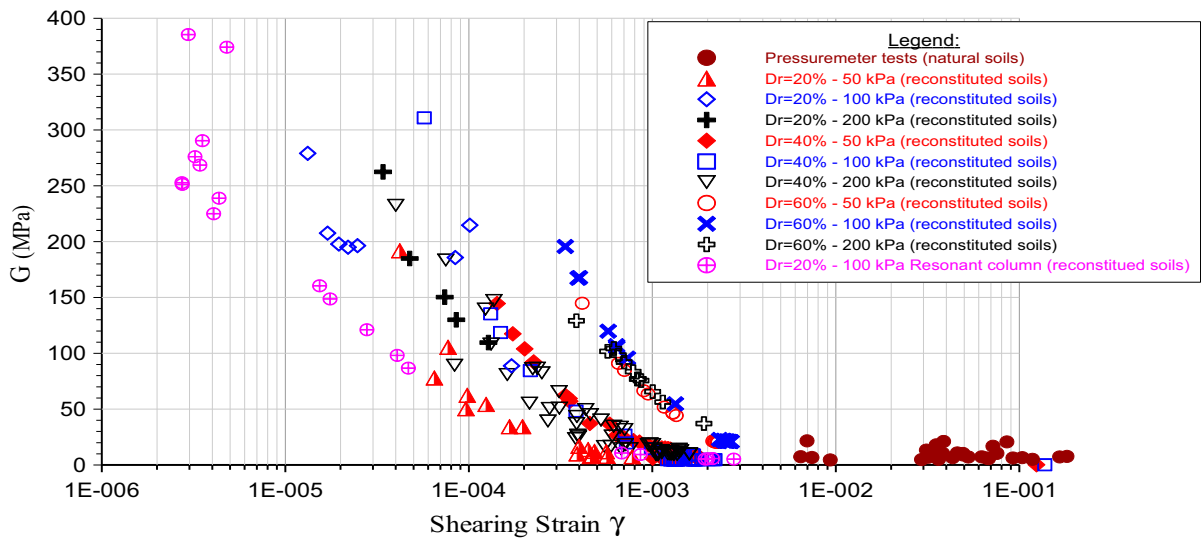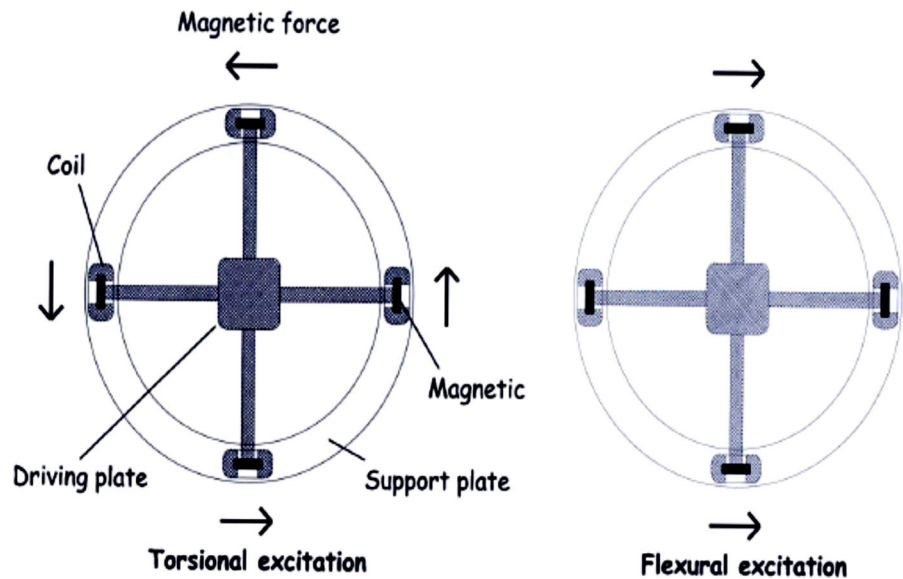**Fig. 23** Execution of the Triaxial Cyclic type CU + u tests

Fig. 24 Experimental shear modulus reduction for relative densities Dr = 20, 40 and 60% with different confinement (50, 100 and 200 kPa)

Fig. 25 Vibration, in torsion or flexure

amplitude of vibration are made. The resonant column method was first developed by Japanese engineers Ishimoto and Iida (1937), has been used to measure the dynamic response of soils (Drnevich et al. 1978; Hardin et al. 1994; Hardin et al. 1994; Laird and Stokoe 1993).

The resonant column test is performed by applying a sinusoidal torque via an electromagnetic drive system to the specimen. The drive system consists of a four-arm rotor that has a permanent magnet fitted to the end of each arm and a support cylinder to which four pairs of wire coils are fitted. During sample preparation the drive plate is connected to the specimen and the height of the support cylinder is adjusted to allow the magnets to be positioned centrally within the wire coils. After the resonant column specimen has been prepared and consolidated, cyclic loading begun. The loading frequency is initially set at a low value and is then gradually increased until the response (strain amplitude) reaches
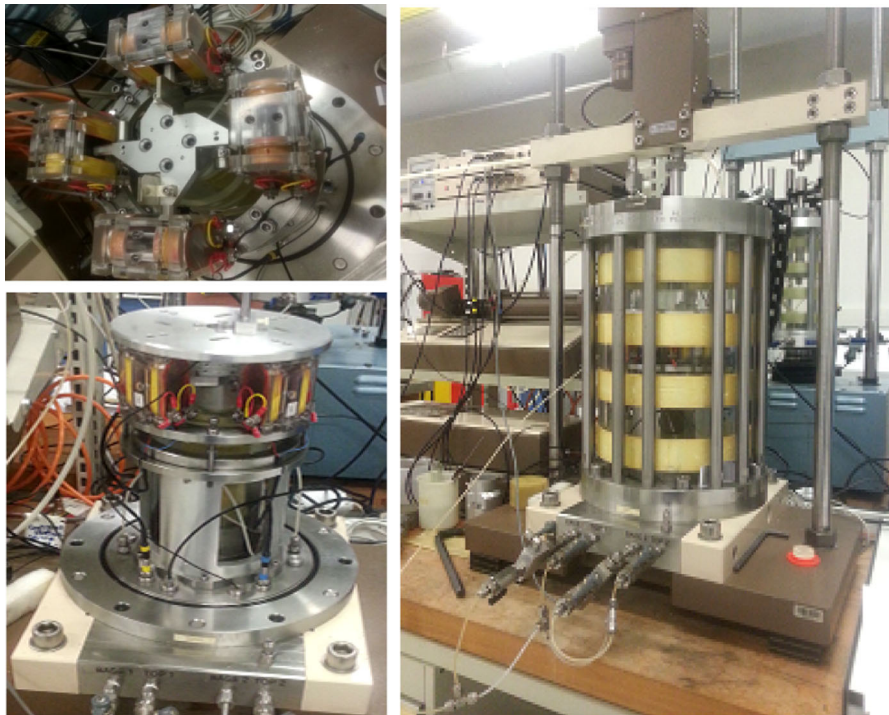
Fig. 26 The resonant column test of the IFSTTAR laboratory

a maximum. The lowest frequency at which the response is locally maximized is the fundamental frequency of the specimen. The fundamental frequency is a function of the low strain stiffness of the soil, the geometry of the specimen, and certain characteristics of the resonant column apparatus (Column 2010).

Typical undrained resonant column tests were performed on the reconstituted soil relatively close to that of the soil in place at a relative density $D_r = 20\%$ with a confinement stress of 100 kPa to generate the degradation curve of the shear modulus of the reconstituted soil. Noting that for a resonance test, in the elastic range ($\gamma < 10^{-5}$), the maximum modulus are related to the propagation velocity of waves in soils. The results of the resonant column tests are presented in Fig. 24.

5 Numerical Analysis

To compare the shear modulus reduction curve $G/G_{max}(\gamma)$ and the increase of damping $D(\gamma)$ with the cyclic shear strain amplitude of Bejaia alluvial

plain soils with the measurement from in situ and laboratory tests on reconstituted soil. Using data resulting from geophysical and geotechnical investigations, calculations are carried out by means of two-dimensional geotechnical difference-element analyses. It consists on numerical identification of these curves using the FLAC^{2D} software (Itasca, V7.0). The non linear elastic model proposed by Ramberg–Osgood (Ramberg and Osgood 1943) and limited by the Mohr–coulomb criterion is used. The experimental curves given by Seed and Idriss (1970) for granular materials and Vucetic and Dobry (1991) for the fine grained soils are used to calibrate the parameters of the model.

5.1 Model Parameters

Ramberg–Osgood hysteresis formulation between shear stress and distortion allows simulating non-linearity even in the elastic domain of materials (Le 2006; Ramberg and Osgood 1943; Saci 2011). This has the advantage of better explaining the notion of degradation of the shear modulus through its

mathematical form and of naturally reproducing the damping of materials under cyclic stresses.

The classical Ramberg–Osgood equation is:

$$\gamma - \gamma_c = \frac{1}{G_{max}} \left[1 + \alpha \left(\frac{|\tau - \tau_c|}{n\tau_y} \right)^{r-1} \right] (\tau - \tau_c) \qquad (5)$$

where n = 1 during the first loading and n = 2 afterwards; τ_c and γ_c are respectively the stress and the shear deformation during the last change of direction of loading; G_{max}, initial tangent shear modulus; r, α, model parameters; τ_y, maximum deviator; γ_y is connected to τ_y by the relation $\tau_y = G_{max} \cdot \gamma_y$.

5.1.1 Parameters of the Ramberg–Osgood Model

5.1.1.1 The Parameter r The constant r (r ≥ 1) is a parameter controlling the rate of increase of the non-linearity of the stress–strain relationship. If r = 1, we obtain a linear elasticity relation:

$$\gamma - \gamma_c = \frac{1}{G_{max}} [1 + \alpha](\tau - \tau_c) \qquad (6)$$

If r tends to infinity, the Ramberg–Osgood formulation becomes

$$\gamma - \gamma_c \rightarrow \frac{1}{G_{max}} (\tau - \tau_c) \text{ si } \frac{|\tau - \tau_c|}{n\tau_y} < 1 \qquad (7)$$

$$\gamma - \gamma_c \rightarrow \infty \text{ si } \frac{|\tau - \tau_c|}{n\tau_y} > 1 \qquad (8)$$

Here an elastoplastic behavior is found. The Ramberg–Osgood formulation is therefore capable of describing linear, nonlinear, or elastoplastic elastic relationships between shear stress and distortion.

5.1.1.2 The Parameter α The constant α is therefore defined by the ratio between the maximum shear modulus and the shear modulus at $\tau = \tau_y$. The parameter α, for the first loading, is defined at $\tau = \tau_y$ by:

$$\left(\frac{G}{G_{max}} \right)_{\tau - \tau_c} = \frac{1}{1 + \alpha} \qquad (9)$$

$$\alpha = \left(\frac{G}{G_{max}} \right)_{\tau - \tau_c} - 1, \quad \alpha > 0 \qquad (10)$$

5.1.1.3 The Parameter τ_y The parameter τ_y denotes the maximum shear strength and is defined by:

$$\tau_y = \sigma'_{zz} \tan \varphi + c \qquad (11)$$

where σ'_{zz} vertical effective stress, φ is friction angle and c cohesion.

In summary, the parameter r determines the degree of non-linearity. The two parameters, α and τ_y influence the relationship between the secant modulus and the maximum modulus at "break", in the sense of the theory of classical plasticity.

5.1.1.4 The Damping Coefficient D The appearance of a hysteresis loop during a closed loading cycle shows a dissipation of energy in the material (Fig. 27). The term damping is used to describe the physical phenomenon of conversion of energy from deformation into heat. The coefficient D damping is defined by the energy dissipated by the material during a cycle closed by the formula:

$$D = \frac{\Delta W}{4\pi W} \qquad (12)$$

where ΔW, energy dissipated during a loading cycle; W, elastic energy stored at the end of the cycle.

The damping coefficient is defined by:

$$D = \frac{2}{\pi} \frac{r - 1}{r + 1} \left(1 - \frac{G}{G_{max}} \right) \qquad (13)$$

The maximum damping is simply expressed by the parameter r

$$D = \frac{2}{\pi} \frac{r - 1}{r + 1} \qquad (14)$$

Finally, the damping of the material varies with the degradation function:

$$\frac{D}{D_{max}} = H\left(\frac{|\tau - \tau_c|}{n\tau_y} \right) = \frac{\alpha \left(\frac{|\tau - \tau_c|}{n\tau_y} \right)^{r-1}}{1 + \alpha \left(\frac{|\tau - \tau_c|}{n\tau_y} \right)^{r-1}} \qquad (15)$$

The main objective of our study is to obtain the curves of the evolution of dynamic parameters of the alluvial plain of the port area of Bejaia. To do this, we simulate a shear test on a soil column subjected to cyclic loading. The proposed procedure is to identify first, from the available geotechnical and geophysical tests of the studied soil. The physical and mechanical parameters determined from the geotechnical tests

Fig. 27 Hyperbolic law—monotonous solicitation—hysteresis loop (Le 2006)
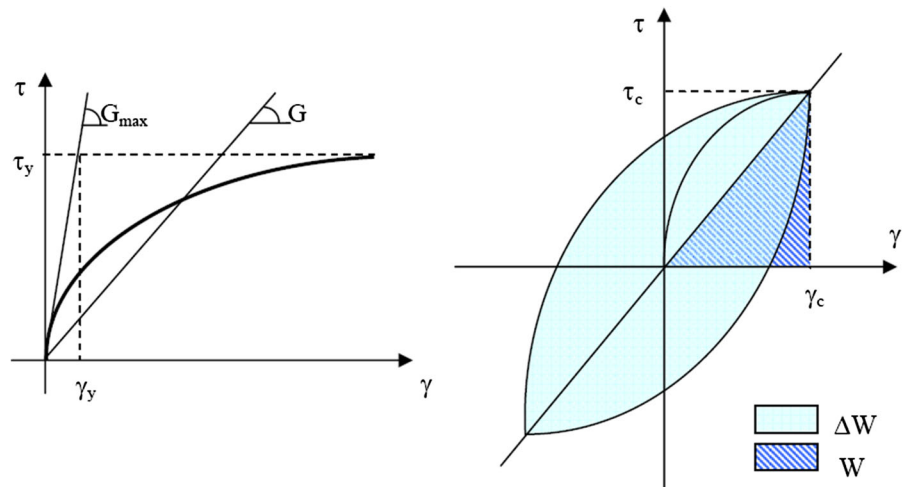
namely: the angle of friction φ, the density ρ, the cohesion C as well as the plasticity index Ip, whereas the maximum shear modulus is defined in situ from the geophysical surveys (Down hole tests) carried out in harbor area of Bejaia (CTELAB 2008).

The numerical simulation is composed of a column of 30 elements made up of the soil to be studied. Firstly, the horizontal nodes are fixed, once the static equilibrium is reached; the nodes following the horizontal are unlocked and those of the base are fixed. The horizontal stresses, which should be proportional to the vertical stresses by a factor near k_0, are then recovered and applied to the limits. They are calculated from the following relationships:

$$\sigma_{yy} = \rho\,g\,h \tag{16}$$

$$\sigma_{xx} = k_0\sigma_{yy} \tag{17}$$

$$\sigma_{zz} = \sigma_{xx} \tag{18}$$

To simulate the arrival of a seismic wave front and more particularly the shear waves, it will be applied to the base of the column a horizontal acceleration. The wave will then propagate itself in the column. The acceleration can be either that of a real earthquake or a mono harmonic sinusoid with modulated amplitude.

Appropriate choices of soil properties are necessary in order to have an accurate simulation in the numerical modeling. The soil properties are determined from laboratory and in situ tests.

The average values of the actual parameters (c′ and φ′) used in the calculations are determined from the results of consolidated drainage (CD) direct and triaxial shear tests. The tests are performed on undisturbed saturated soil samples. However, the average values of the effective parameters (c′ and φ′) determined from the consolidated drained type (CD) triaxial results on reconstituted samples are relatively close also to those made on undisturbed soil samples in place (Sadaoui and Bahar 2017).

The parameters of the Ramberg–Osgood law, r and α, are identified by means of a practical procedure developed by comparing the curve G/G_{max} given by this law and the experimental curve G/G_{max}. The evaluation of maximal shear modulus is determined from geophysical methods, often used to characterize the dynamic soil properties of the subsurface, in particular down hole test.

Because no experimental curves G(γ) and D(γ) are available for Algerian soils, the identification of the model parameters, r and α, is based on the empirical curves proposed by Seed and Idriss (1970) for granular materials and Vucetic and Dobry (1991) for clays. In second step, a cyclic behavior of a soil profile is simulated using the numerical code FLAC, taking into account Ramberg–Osgood law limited by the Mohr–Coulomb criterion. The angle of friction φ, the cohesion c, and the effective mean stress p′ are defined from geotechnical investigations.

Dynamic site response analysis is performed considering a 2D finite difference model. The soil deposit is assumed to be made of several horizontal layers, which are horizontally homogenous and of infinite horizontal extent soil column extending from the ground surface to bedrock. The soil layers are

characterized by their physical and mechanical properties of the model taking into account in this study. The mesh of the soil profile is 30 rectangular elements of one meter in thickness and width. The mesh size for soil column model is selected to ensure accurate wave transmission. Earthquakes or mono-harmonic sinusoidal functions are assumed to generate only horizontal motion at the soil–bedrock interface, and to result into shear waves that propagate vertically in the soil layers. Figure 28 shows the model prediction for four acceleration amplitude. The results illustrate the potentiality of the model to fit the non linear stiffness of soil. It can be noted that with the increase of the amplitude of the loading loops are more and more tilted towards the horizontal what brings to light the degradation of the shear modulus. The area of the

hysteresis loop on the other hand increases proportionally with the development of the amplitude of loading, what explains the increase of the damping ratio D according to the distortion γ.

In Harbor zone of Bejaia site, the Ramberg–Osgood constants are identified using the empirical curves of Seed and Idriss (1970) for sands and Vucetic and Dobry (1991) for clays. Then, a simulation with FLAC program is performed taking into these parameters for the Ramberg–Osgood law and the Mohr–Coulomb criterion. Figures 29 and 30 shows the best fit Ramberg–Osgood curve and compares the calibration curve to those obtained by numerical calculation using FLAC code. The soils are muddy silty sand and Muddy sand of Bejaia.
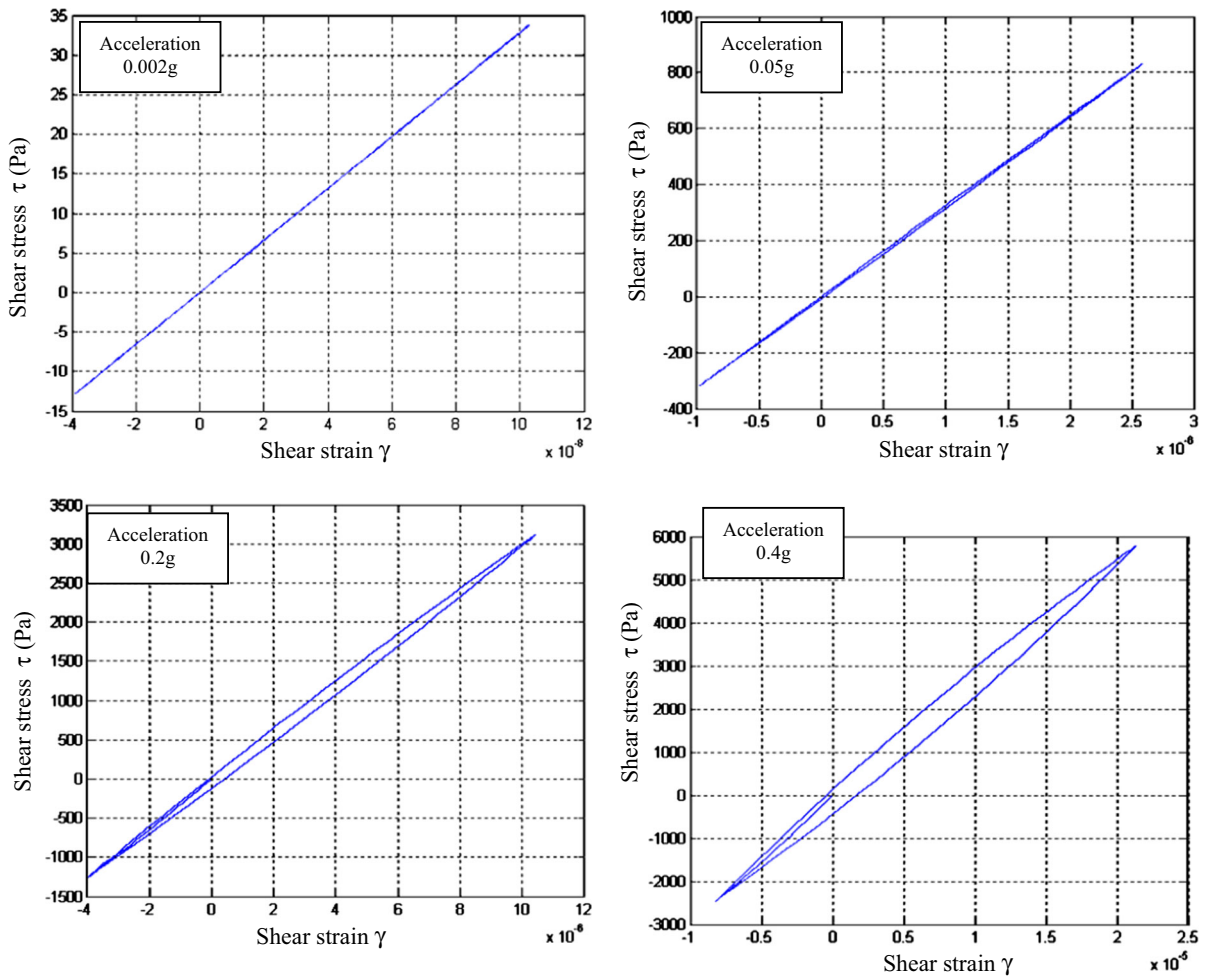

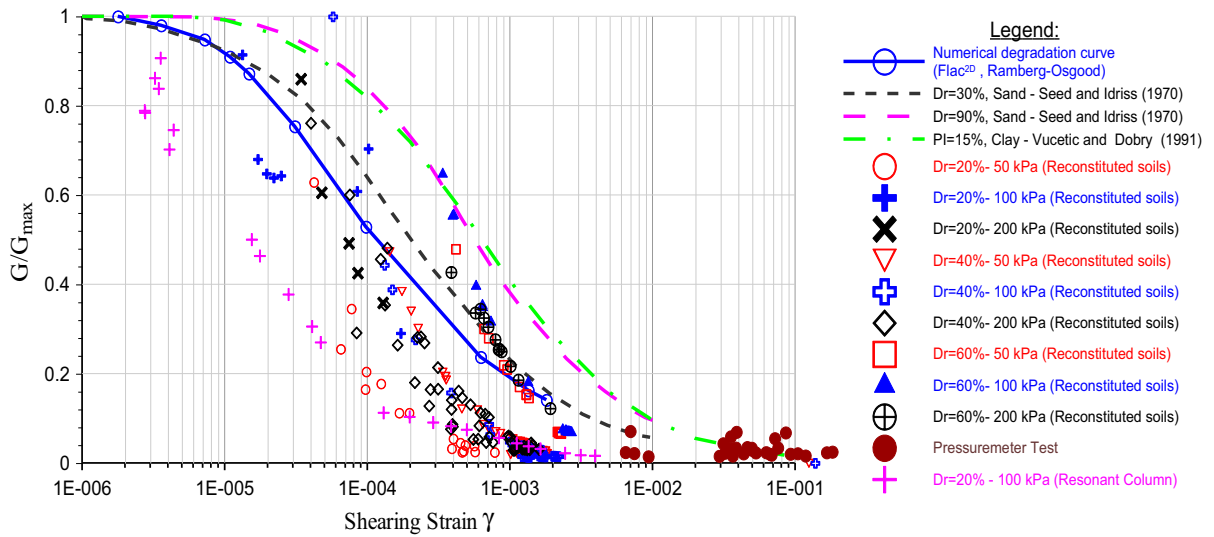**Fig. 28** Shows the model prediction for four acceleration amplitude

Fig. 29 Comparison of results shear modulus reduction variation with distortion deformation
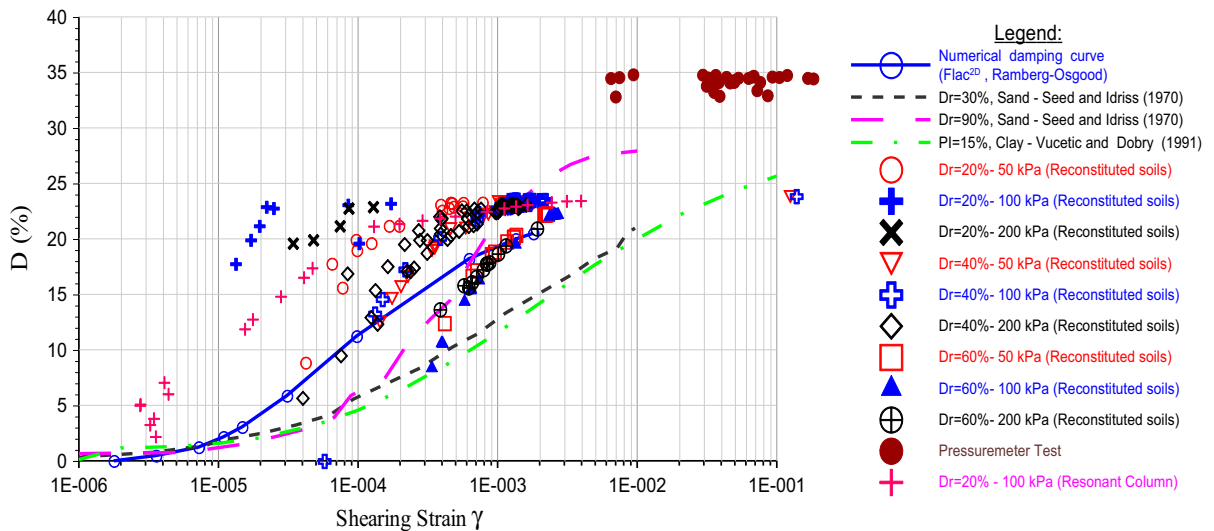

Fig. 30 Comparison of results damping D (%) variation with distortion deformation

In general, laboratory test results were in reasonable agreement with numerical trends. The shear modulus and damping rate evaluated showed containment dependence. The reduction of the cyclic triaxial test module, resonant column and data pressuremeter appear to be relatively close to the numerical result, and in comparison with other data in the literature. The model presented by Ramberg–Osgood is applicable to the muddy silty sand and muddy sands of Bejaia.

6 Conclusions

The Procedures for cyclic triaxial and resonant column testing soil were performed to estimate the shear modulus decreases and damping curves in the laboratory. Results on muddy silty sand and muddy sand in the Bejaia harbor area have been found to be appropriate in comparison to the literature and models proposed by other authors. In general, the numerical results were reasonably consistent with laboratory test

trends. The shear modulus and damping rate evaluated showed containment dependence. The shear modulus decreases and damping rate evaluated of the numerical results appeared to be relatively close to cyclic triaxial test, resonant column and data obtained from pressuresmeter, and in comparison with other data in the literature. The model presented by Ramberg–Osgood is applicable to the muddy silty sand and muddy sands of Bejaia.

The shear modulus decreases and damping rate evaluated for sand-fines mixtures on reconstituted soils can be predicted based on plasticity index and natural grain size curve and there is good correlation between the predicted and literature data.

The modulus degradation behaviour of in place soils and those on reconstituted soils of sand-silty clay mixtures at the same PI is relatively close. The degradation curves of sand-fines mixtures show less stiffness than those of pure clays at same PI. For sand-silty clay mixtures on reconstituted soils containing low plastic fines, the normalized shear modulus (G/G0) curves literature data are stiffer compared for curves obtained for alluvial plain of Bejaia using data resulting field and laboratory tests.

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