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Technical session 2e: Marine and transportation geotechnical engineering

Séances techniques 2e: Géotechnique marine et de transport

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1 INTRODUCTION

The wide variety of topics covered in the 32 papers of this session shows the broad interest of geotechnical engineers in the-marine and transportation geotechnical fields. The papers were categorized by their primary field of application. Three fields were identified: offshore geotechnical engineering, pavements (road and airport) and railways. Tables 1, 2 and 3 list the papers associated to these three field, giving for each a brief description of the objectives of the paper, type of structure and/or soil type, tools used in analysis (numerical, experimental, empirical), material model used and the main conclusions reached.

To set up the session and bring perspective around the papers, and to promote some exciting discussion, this general report briefly reviews some of the recent advances in the state-of-the-art associated with offshore geotechnical engineering, pavements and railways. For both the marine (offshore) and transportation geotechnical engineering contributions to this session, the papers can be categorized under four main topics: soil characterization, physical modeling, theoretical modeling and monitoring. No less than 10 of the 32 papers deal with physical modeling, eight with soil or/and material characterization, five with modelling and eight deal with monitoring. In fact, only a few papers present case studies of foundation design.

2 OFFSHORE GEOTECHNICAL ENGINEERING

The topic of offshore geotechnical engineering is extremely vast. Only three main aspects are approached in this short review paper: foundation design, anchors in clay and offshore geohazards.

Most of the papers to this session (Table 1) contribute to one or several aspects of the topics covered below, except the last paper in the table which probably should have been placed in a session about determination of soil characteristics. The papers dealing with offshore geotechnical engineering can be classified as follows:

Topic	Authors
Foundation design, especially performance under cyclic loading	Hamre et al. Valore and Zicarell (quay wall)
Anchors in clay	Singh et al. Micic and Lo
Offshore geohazards	Strout and Sparrevik Puech et al. Kliner and Grozic
Pipelines	Cheuk et al Vanden Berghe et al
Modelling in the laboratory, centrifuge and in situ	Laue et al. Tufenkjian and Thompson Elmi and Favre Kikuchi et al.

2.1 *Foundation design*

Cyclic loading generates pore pressure and reduces the effective stresses in soils, causing average and cyclic shear strains that increase with number of cycles (Fig. 1). Knowledge about the behaviour of soils under cyclic loading is essential for the foundation design of the different offshore structures, including fixed offshore oil and gas platforms, near shore liquid natural gas (LNG) terminals, anchors for floating structures, harbours, breakwaters, storm surge barriers, etc. Wave loading will normally have load periods of the order of 10 s or more. The fundamental behaviour of soils is independent of the load period, and the general principles apply independently of period. The numerical values are however rate-dependent and will be a function of the load period.

The summary of key issues for offshore foundation engineering below is based on the contributions of Andersen (1991) and Andersen (2004).

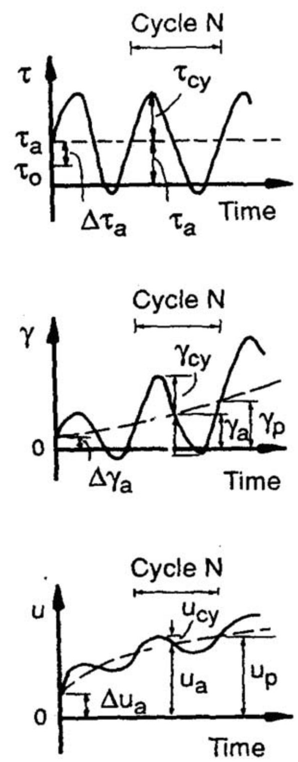


Figure 1. Shear stress (τ), shear strain (γ) and pore pressure (u) under cyclic loading (N= number of cycles, subscripts cy= cyclic, a=average, o=initial, p= permanent) (Andersen, 1991)

Table 1. Summary of TS2e Papers Related with Marine (Offshore) Geotechnical Engineering

Authors	Topic	Objectives	Structure and/or soil/material type
Badv	Chemical and mineralogical characteristics of lake deposits	Investigate characteristics of soil specimens 100m down into lakebed.	For bridge foundation, Lacustrine clay deposits.
Cheuk et al.	Pipelines	Establish deformation mechanism during pipe uplift in sand.	Pipeline in both dense and loose sand.
Elmi & Favre	Theoretical simulation fo CPT cone resistance	Simulation of q_c is done form seismic measurement on marine soils.	All soil types.
Hamre et al..	Offshore foundation	Prediction of pore pressure development under wave loading and its dissipation through a drainage system.	Two concrete platforms on sand and overconsolidated clay.
Kikuchi et al.	Lightweight material for waterfront applications	Study the permeability characteristics of lightweight treated soil mixed with air foam (LWS).	Natural marine clay and treated mixtures.
Kliner & Grozic	Offshore geohazards (hydrates in offshore soils)	Simulate hydrates; correlate hydrate content and dielectric response of specimens.	Ottawa sand with synthetic refrigerant hydrates (R-11).
Laue et al.	Centrifuge modelling for offshore applications	Model soil samples with very low densities in centrifuge.	Sand only
Micic & Lo	Suction anchor	Study strengthening effect of electrokinetic treatment on clays.	Model steel cylinder embedded in natural marine clay
Puech et al..	Offshore geohazards (slope instability)	Geotechnical properties of sediments of the continental slope of the Gulf of Guinea in water depths greater than 450m.	Continental slope of the Gulf of Guinea (Angola), softclay.
Singh et al..	Suction anchor (pile)	Study pore water pressure changes and suction development during vertical pullout of suction anchor (superpile).	Suction anchors embedded in soft saturated clay as alternative to piles for TLP.
Strout & Sparrevik	Offshore geohazards (pore pressure at depths in seabed sediments)	Development of a multilevel piezometer system capable of pore pressure measurements at several discrete depths within a single boring, with water depths of 1500 m.	Field verification in seabed sediments in Norwegian fjord.
Tufenkjian & Thompson	Minicone test for offshore applications	Study penetration resistance of minicone at shallow penetrations.	In sand.
Valore & Ziccarelli	Rehabilitation of quay wall	Case history of quay wall on the everge of collapse and its rehabilitation.	Reinforced concrete caissons filled with lean concrete over dense sands.
Vanden Berghe et al.	Pipelines	Simulations of soil behaviour during uplift displacement of a pipeline by FEM.	Pipeline buried in very loose sand.

Table 2. Summary of TS2e Papers Related with Transportation Geotechnical Engineering – Pavements (roads and/or airports)

Authors	Topic	Objectives	Structure and/or soil/material type
Edil & Sawangsuriya	Application to pavements: monitoring	Assess soil stiffness for construction quality control of earthworks.	Earthwork
Erlingsson	Pavement: material characterization, modelling and validation	Mechanical properties of constituent materials to use in modelling.	Thin surface dressed pavement

Table 1. Continuation: Summary of TS2e Papers Related with Marine (Offshore) Geotechnical Engineering

Tools used in investigation	Tools used for analysis	Material models	Main Conclusions
Pneumatic pore water squeeze apparatus and chemical and mineralogical tests in laboratory	NA	NA	Chloride and sodium contents, pH, salinity and electrical conductivity were almost uniform in depth and consistent with lake water; calcium showed a gradual increase with depth.
Model test in plane strain calibration chamber in laboratory; digital cameras.	Image analysis (PIV), photo-grammetry; limit equilibrium analysis; dilatancy theory.	Equations of failure with dilatancy.	In dense sand, the failure deformation at peak resistance comprised a distributed shear zone bounded by planes inclined at the angle of dilation above the pipe; beyond peak, there is formation of narrow vertical shear bands leading to strain softening; particle size did not affect peak uplift resistance or mobilisation distance for the study conditions.
Seismic tests (seismic penetrometer and SASW) insitu; pressio-penetrometer.	FEM-GEFDYN code	Elastoplastic with hardening.	It is possible to obtain tip resistance from seismic measurements and identification of marine sediments; need further validation and an analysis of uncertainties.
Cyclic and static simple shear and triaxial tests in lababoratory, Boreholes and CPT's in situ.	FEM-CYCPOR2 2D analysis.	2D transient pore pressure annalysis.	Significantly different behaviour of sand under cyclic and static loading. Prediction of pore pressures in top sand, installation of foundation ribs and use of drainage system provided simple and cost-saving solutions.
Permeability tests in triaxial apparatus and constant rate consoli-dation tests; X-ray in the laboratory.	NA	NA	Permeability of LWS is controlled by clay mixed with cement, assuming that air voids in the material are impermeable compressible media.
Theta probe in laboratory, measuring unhydrated volumetric water in comparison with initial hydrate formation percentages.	Dielectric theory.	NA	The bulk dielectrical response exhibits a decreasing trend with increase of pore space synthetic R-11/H_2O hydrate content. Further validations to other soil types, densities and salinities are necessary to pass from laboratory to field applications.
Drum centrifuge; model soils tested with PT and shallow foundation	NA	NA	Sedimentation of sand spread over free water is appropriate to create samples in very loose state, contrarily to the dry pluviation technique where minimum density achieved is 25%.
Physical model in laboratory, dia = 200 mm. L = 50 cm	NA	NA	Pullout capacity of a cylindrical steel foundation model increased five times due to electrokinetic treatment. This treatment induced a change in failure mechanism, the cementation of soil particles and the bonding between soil solids and the steel objects.
Physical, chemical and mechanical tests in laboratory; borings, CPT and vane tests in situ	NA	NA	Geotechnical material characterisation from 10 sites of continental slopes of West Africa at depths between 450 to 1500m shows similar properties (physical characteristics, undrained shear strength and in situ stresses; need to correlate compressibility and shearing resistance to geological history and microstructure.
Laboratory tests on model anchor in clay.	NA	NA	Under short term tensile loading, suction development contributes to the pullout capacity. Under long term tensile loading, resistance depends on the dead weight of the foundation and shearing along the interior and exterior of the superpile wall only. For maximum cyclic loads greater than the dead weights and external skin friction, pullout could arise as number of load cycles increase.
Testing in grout columns and pressure containers in laboratory; two installations of multilevel piezometer system in situ.	NA	NA	Operational equipment capable of multi-level pore pressure measurements within a single subsea installation. Can operate down to 1500 m water depth and down in borehole to 200 m; penetrations depend on soil type, typically 10 to 20 m in soft marine clays; Measured pore pressures were correct.
Model in test bed and minicone in laboratory, comparesons with CPT	NA	NA	Standard CPT gives greater tip resistance and similar critical depth; results may be affected by boundary effects.
Boreholes, SPT, monitoring of movements of structure.	FEM-Plaxis code	Not referred.	Monitoring of movements of structure was crucial for diagnosis of its safety conditions; stabilisation work should take into account eventual adverse effects during construction.
Existing data from undrained triaxial tests in laboratory.	FEM-Plaxis code; empirical equations.	Hardening soil model in Plaxis code.	Very loose sand state generate a flow-around mechanism (local failure) during uplift, contrary to dense sands where a wedge failure mechanism until surface develops; pipeline uplift in loose sands is likely to occur in the quadrant ± 30º from the vertical.

Table 2. Continuation: Summary of TS2e Papers Related with Transportation Geotechnical Engineering – Pavements (roads and/or airports)

Tools used in investigation	Tools used for analysis	Material models	Main Conclusions
Field: SSG & nuclear gauge.	Empirical equations	NA	SSG with independent moisture content measurement is appropriated for construction quality control of earthworks.
Lab.: RLT Field: HVS and instrumented pavement	MLEST; FE	Linear and non linear elastic	Better predictions using non linear than linear analysis for base material; RLT simple power law agrees with rutting measurements in field

Table 2. Summary of TS2e Papers Related with Transportation Geotechnical Engineering – Pavements (roads and/or airports)

Authors	**Topic**	**Objectives**	**Structure and/or soil/material type**
Gnanendran	Airport: physical model and monitoring	Investigation (using a physical model) of the effects of combined cyclic vertical and horizontal loading on the pavements.	Unsealed airfield pavement
Gomes Correia et al.	Application to pavements: material characterization	Effects of stress state and strain level in Young's modulus, as well as the stress-strain behaviour during a large number of cycles.	Only gravel
Hayano et al.	Pavement: physical models and monitoring system, modelling	Stress distribution in asphalt pavement subjected to roller loads	Only gravel; 50mm of asphalt layer on gravel.
Kodikara & Ranjith	Pavements: material characterization	Examine salt migration and crystallization in stabilised pavement materials.	Only crushed basaltic rock stabilised with cement.
Lee et al.	Pavement: monitoring	Overview of RDD and its key features; Benefits of continuous deflection profiles.	Rigid pavements; rehabilitated concrete pavements; flexible pavements.
Sargand & Kim	Pavement (Ohio test road): monitoring, case histories	Investigation of early longitudinal cracks and rutting in Ohio Test Road.	Ohio test road.
Vallejo & Chik	Application to pavements: material characterization	Application of fractal theory to evaluate changes in the size distribution of a sand under different crushing levels simulated in a ring shear apparatus; Effects of sand fragmentation in hydraulic conductivity and on the shear strength.	Only crushed aggregates from granite.
Wijeyakulasuriya et al.	Pavement: material characterization	Undrained behaviour of well–graded aggregates; Rank materials based on plastic strain rate criteria	Flexible pavement with 40mm asphalt.

Table 3. Summary of TS2e Papers Related with Transportation Geotechnical Engineering – Railways

Authors	**Topic**	**Objectives**	**Structure and/or soil/material type**
Abdelkrim et al.	Railways and pavements: modelling	Predict permanent settlements under large number of cycles	Only ballast
Ed Calle et al.	Roads & Railways: modelling, monitoring	Approach to predict long-term settlements based in short time observations.	Embankments
Kopf & Adam	Railways: modelling, case histories	Consider dynamic effects in design of rail track systems.	Ballast-less railroad (Vienna metro); Ballast rail track of a high-speed railway.
Momoya & Sekine	Railways: physical model and modelling	Influence of the stiffness and thickness of roadbed on the resilient and permanent deformations of roadbed and subgrade under moving wheel loads.	Railway ballast structure without and with asphalt roadbed.
Nurmikolu & Kolisoja	Railways: material characterization	Evaluate long term durability of insulation boards used in the frost protection of rail tracks.	Ballast rail track with frost insulation boards under ballast
Queiroz & Macari	Railways: physical model	Study load-deformation behaviour of rail track systems.	Railway ballast structure with 3 different railroad ties.

Table 2. Continuation: Summary of TS2e Papers Related with Transportation Geotechnical Engineering – Pavements (roads and/or airports)

Tools used in investigation	Tools used for analysis	Material models	Main Conclusions
Lab.: Physical model test	Empirical equations	NA	Rutting and horizontal deformation develops under load cycles; The inclusion of geogrid reinforcement in pavement structure can decrease these deformations.
Lab.: Large precision cyclc triaxial test	Generalised elastic theory.	Hypo elastic model.	Effects of recent strain history on the small strain Young's modulus (E_0)are practically negligible.; E_0 is essentially a unique function of the stress in the same direction while it is rather independent of the other orthogonal stresses; Increase of tangent and secant E during global reloading of triaxial compression with an increase in the strain until the plastic strain rate starts significantly increasing .
Lab.: series of model tests.	FE	Isotropic linear elastic.	New monitoring system to measure vertical and shear stresses; Agreement of measurement results with numerical results using linear elastic model, when shear force on the ground surface are taken into account.
Lab.: Simulation of capillarity rise of salt solutions in cylindrical specimens.	Thermodynamic theory.	Empirical equations.	Digital recording in testing is necessary to have more accurate capture of initiation and change of crystallization; Salt crystallization depends on: salt water concentration, atmospheric climate, pore structure of material and depth of water table.
Field: FWD, RDD	NA	NA	RDD powerful tool to identify changes in pavement stiffness to help in project level studies and rehabilitation schemas.
Field: Distress survey, FWD, Transverse profiles, DCP.	MLEST	Linear elastic.	Important role of moisture in rutting and cracking; Augers or spreaders in pavers are at the origin of straight line cracks in pavements; Longitudinal cracks top-down; Rutting develops in base layers and subgrades.
Lab.: ring shear apparatus.	Fractal theory.	NA	The change particle size distribution from non-fractal to fractal decrease hydraulic conductivity and peak internal friction angle of sand.
Lab.: RLT & WT Field: FWD	Database with more than a decade.	MLEST (FWD backanalysis).	Inability of current specifications to rank materials; need of performance-based tests; Role of plastic fines needs further study.

Table 3. Continuation: Summary of TS2e Papers Related with Transportation Geotechnical Engineering – Railways

Tools used in investigation	Tools used for analysis	Material models	Main Conclusions
Lab.: Physical model test of ballast under cyclic loading (fix point loading); Data available from cyclic triaxial tests.	FEM – CESAR-LCPC	Permanent settlements vs. number of cycles, in function of stress path.	The developed numerical tool still need more experimental results to be validate.
NA	Bayesian framework.	NA	An alternative approach for determining variances of uncertainty margins of long term settlements predictions by using Bayesian theory taking into account monitoring data; Ongoing research with important implications in design of a monitoring strategy.
Field: Ballast-less instrumented metro-line section; Instrumented test section of a high-speed ballast rail track.	Model considering a flexible beam (finite length) resting on continuous spring-dashpot elements.	Viscoelastic.	Design model developed to determine dynamic response of different rail track systems under load moving with constant speed; Further developments are needed to account non-linear material behaviour of materials like synthetic pads.
Lab.: Physical model test (moving-wheel loading); image processing results to represent displacements vectors.	FEM – 3D	Not described.	Operational laboratory facility to simulate the travelling trainload on railway track, proved to give more realist results than fixed-point loading tests; Important role of asphalt concrete layer to reduce deformation of roadbed and subgrade; Resilient deformations independent of permanent deformations under cyclic loading.
Lab.: Cyclic loading tests and static compression tests. Field: Specimens taken from different rail track sessions.	NA	NA	Good long performance of insulation boards needs good strength properties and high resistance to water absorption. Dimensioning value for the thermal conductivity and compressive strength key parameters to delivery insulation boards.
Lab.: Full scale physical model, with internal and external instrumentation (fix point loading).	Empirical equations.	Empirical equations.	Prestressed concrete railroad ties underwent less deformation than wood and steel ties under same loading conditions; Establishment of empirical equations relating deformation as a function of number of cycles.

Table 3. Continuation: Summary of TS2e Papers Related with Transportation Geotechnical Engineering – Railways

Authors	Topic	Objectives	Structure and/or soil/material type
Vinogradov et al.	Railways: physical model	Modelling of the serviceability of railway embankments on geotechnical centrifuges under dynamic loads.	Railway embankment
Wilkinson et al.	Application to railways: material characterization	Study the physical-chemical mechanisms and factors affecting stabilisation performance in order to help design of cementitious slurry components.	Railways subgrades

2.1.1 *Bearing capacity*

The geotechnical design analyses have to ensure that the soil has sufficient capacity to carry the weight of the structure and the cyclic loads with an adequate safety against excessive deformations. The capacity under cyclic loading may be different under cyclic loading than under static loads. The cyclic capacity is often smaller than the monotonic capacity, but the difference will depend on the number of cycles and the composition of the cyclic amplitudes, see e.g. Andersen (2004) and Dyvik et al. (1989).

2.1.2 *Cyclic displacements*

Cyclic displacements may cause a serviceability problem and may induce stresses in structural elements imbedded in the soil or connections to the structure, like oil wells, risers and pipeline connections for offshore platforms. Cyclic horizontal displacements and cyclic rotation will occur during storm. Yielding may take place, for example in the oil wells underneath a platform if the horizontal displacements are excessive.

2.1.3 *Dynamic foundation stiffnesses*

Equivalent soil spring stiffnesses are necessary for the structural dynamic analyses of offshore platforms. The first resonance periods of a platform should be well below the predominant wave load periods, e.g. 10 s. For platforms in deeper water or with lower structural stiffness, the resonance periods move towards the wave load period, and this may give significant wave load amplification. It is therefore important to make sure that the resonance periods are far enough away from the cyclic load periods.

2.1.4 *Settlements due to cyclic loading*

The weight of the structure and the cyclic loads will cause permanent deformations in the soil beneath and outside the structure. The permanent displacements may cause stresses in structural elements in the soil or connections to the structure, like oil wells, risers and pipeline connections for an offshore platform. For offshore platforms, vertical settlements will also reduce the free-board between the deck and the sea.

The deformations due to cyclic loading may be separated into two components; (1) due to increased permanent shear strains due to the cyclic loading and (2) dissipation of cyclically induced pore pressure.

2.1.5 *Soil reaction stresses*

The structure must be designed for the stresses from the soil, both due to the static and the cyclic loads. These stresses may redistribute due to degradation of the soil modulus under cyclic loading.

2.1.6 *Required cyclic soil parameters*

The following soil parameters are needed to analyse the foundation design aspects of offshore structures:

- Cyclic shear strength
- Cyclic shear modulus
- Damping
- Permanent shear strain due to cyclic loading
- Pore pressure generation
- Recompression modulus

These soil parameters can be determined from triaxial and DSS laboratory tests with various combinations of average and cyclic shear stresses. The recompression modulus can be determined either from (1) cyclic direct simple shear (DSS) tests with subsequent drainage of the cyclically induced pore pressure, or (2) simplified as 2/3 of the reloading modulus from an oedometer test (Yasuhara & Andersen 1991).

A convenient way to interpret and present the data from cyclic laboratory tests is to plot contour diagrams of average and cyclic shear strains and permanent pore pressure as functions of average and cyclic shear stresses (Fig. 2). Such diagrams contain the information required to derive cyclic shear strength and deformation characteristics for foundation design of structures subjected to combined static and cyclic loading. Data published on Drammen Clay for various overconsolidation ratios (Andersen, 2004) can be used to (1) understand soil behaviour under combined static and cyclic loading, (2) provide a framework for planning and interpretation of cyclic laboratory tests for actual projects, thus reducing the required number of tests in actual projects, and (3) provide data to establish the parameters for foundation design in feasibility studies before cyclic soil data on site specific soil are available.

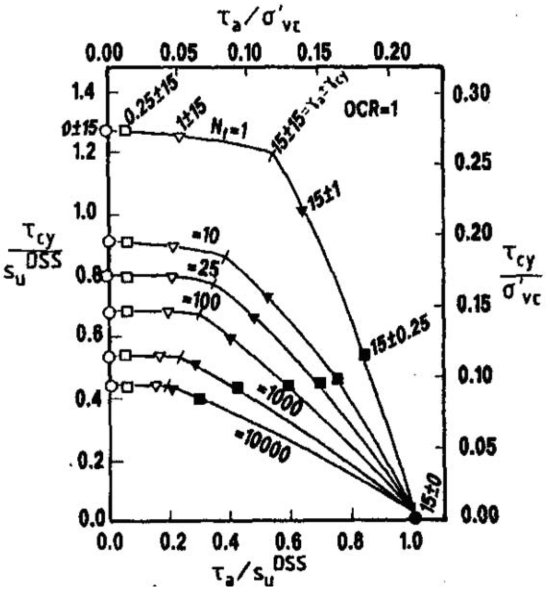


Figure 2. Number of cycles to failure, N_f, and shear strains at failure, $\gamma_{a\pm}\gamma_{\chi y}$, in simple shear tests (DSS) on Drammen clay with overconsolidation ratio of 1 (Andersen, 2004)

Cyclic data and initial shear modulus of a number of clays with different plasticity have been collected and plotted in addition to the Drammen Clay data (e.g. Andersen 1991; 2004). The plots show a tendency for the shear stresses at failure for a given number of cycles to increase with increasing plasticity index when the shear stresses are normalised with respect to the undrained monotonic shear strength. The normalised initial shear modulus clearly decreases with increasing plasticity index and overconsolidation ratio.

Table 3. Continuation: Summary of TS2e Papers Related with Transportation Geotechnical Engineering – Railways

Tools used in investigation	Tools used for analysis	Material models	Main Conclusions
Lab.: Centrifuge models of railway embankments.	NA	NA	Important influence of dynamic loads induced by trains on the stability of embankments.
Lab.: Physical chemical and mineralogical tests.	NA	NA	Soil chemistry/mineralogy can be used as a basis for design of cementitious slurry components; Injection procedure, slurry rheology and soil structure affects overall stabilization performance.

Calculation procedures based on such soil data framework have been verified by back-calculations and predictions of prototype behaviour, and by model tests.

The papers in this session by Hamre et al and Valore and Zicarelli present case studies illustrating innovative and cost-effective solutions. In particular, the solution with bottom ribs and drainage system (Fig. 3) for two gravity structures in the Sakhalin II project (Hamre et al, 2005) lead to important savings in the foundation costs.

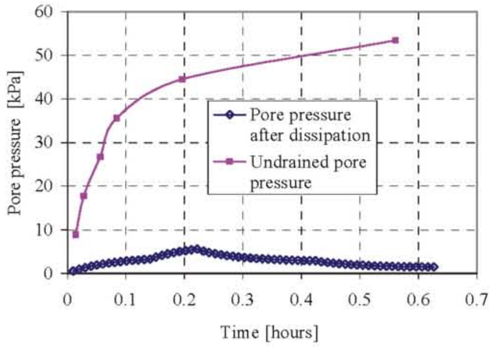


Figure 3. The prediction of excess pore pressure and pore pressure dissipation and the choice of ribs and drainage system resulted in important savings in the foundation costs (Hamre et al, 2005)

2.2 *Anchors in soft clays*

The results of an industry-sponsored study on the design and analyses of suction anchors in soft clays will be presented in two keynote papers at the ISFOG conference in Perth immediately following this conference (Andersen et al 2005 on suction anchors, Murff et al 2005 on vertically loaded drag anchors). These two papers represent the state-of-the-art on anchors in clays. They were prepared jointly by a large group of engineers and scientists working in the field. The two papers evaluated references on over 300 anchors and summarize a number of prediction methods and data related to installation performance and holding capacity of anchors. Research topics with the potential for improving current practice are also identified. The basis of the evaluation was a comparison of predictions of hypothetical cases of various simplified methods as well as a comparison of predictions using these methods with 'ground truth' data from either rigorous 3D finite element analyses or prototype data where available. The bias and uncertainty in predicting the anchor's installation performance and holding capacity were assessed.

2.2.1 *Suction anchors*

A suction anchor is a large diameter cylinder, open-ended at the bottom and closed at the top. Mooring loads are applied by an anchor line usually attached to the side of the caisson. The length to diameter ratio of the caisson is typically six or less. Once installed, the caisson acts much like a short rigid pile and is capable of resisting both lateral and axial loads. The maximum holding capacity is obtained if the chain is attached at a depth where the anchor failure mode is large translational displacements with minimal rotation ('optimum load attachment point') (Andersen and Jostad, 1999, 2002).

The suction caisson (Fig. 4) gets its name from the fact that it is usually installed by applying under-pressure ('suction') to its interior after it is allowed to penetrate under its own weight. The difference between the hydrostatic water pressure outside the cylinder and the reduced water pressure inside provides a differential pressure that acts as a penetration force in addition to the weight. After installation, the caisson's interior is sealed off and vertical loading creates an internal underpressure which in turn mobilizes the end bearing resistance of the soil at the caisson tip.

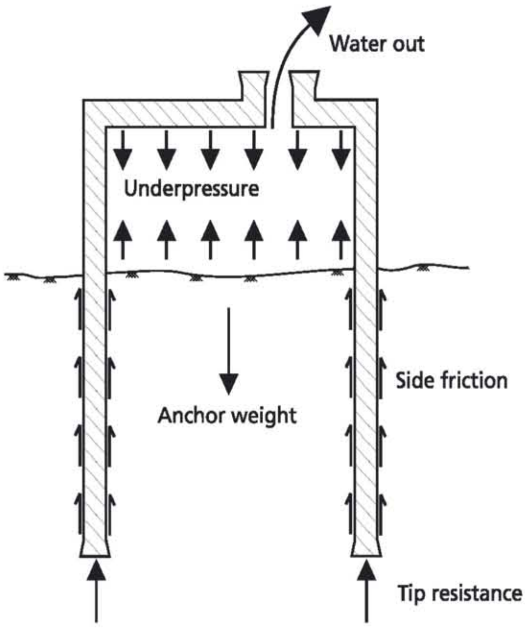


Figure 4. Skirted structure with the forces acting during penetration by underpressure (Andersen and Jostad, 1999)

Proof loading to check the anchor holding capacity after installation is not required for suction anchors because 1) their positioning is well controlled; 2) the foundation design is based on prediction methods calibrated against model test data and detailed numerical analyses, and 3) the soil conditions are normally well documented by *in situ* and laboratory testing. As suction anchors are relatively shallow structures, deep soil borings are not needed, but more detailed soil data are needed at shallow depths than for piles (Andersen et al 2005).

Suction anchors have been used at about 50 locations in water depths to nearly 2000 m during the last decade. Thanks to the willingness of the industry to provide proprietary data, a good set of data on suction anchor applications, experiments and prediction methods has been compiled. This includes detailed installation data for 16 of the 50 locations where suction anchors have been identified.

The industry sponsored study evaluate current practices for predicting the installation performance and capacity of suction anchors, also assessing their simplicity, completeness, sensitivity, practicality, and generality. The methods included those used by the Offshore Technology Research Center (OTRC), the

Centre for Offshore Foundation Systems (COFS), the Norwegian Geotechnical Institute (NGI) and one industry predictor. COFS cooperated with Advanced Geomechanics (AG) on the installation predictions. The study on suction anchors to be published at ISFOG in Perth (September 2005) concluded the following (the conclusions are quoted form the paper by Andersen et al, 2005):

(1) Installation

The penetration resistance of suction anchors is calculated as the sum of the integrated interface shear strength along outer and inner skirt walls and any internal plate and ring stiffeners, and the end bearing resistance of skirt tips, plate stiffeners, ring stiffeners and any changes in anchor diameter. In the case of penetration by underpressure, the required underpressure is calculated as the penetration resistance minus the submerged anchor weight, divided by the inside cross section area beneath the top lid.

As for pile design, both total and effective stress methods may be used to estimate the shear strength along the inside and outside of the anchor. Both approaches require knowledge of the remoulded strength of the soil. The effective stress approach also requires the interface friction angle between caisson and clay and the effective normal stress along the skirt wall. If non-standard wall surfaces are used, such as painted walls, potential reductions in the interface strength or friction angle must be taken into account.

Bearing resistance on the skirt tip, external protuberances, or internal stiffeners is based on bearing capacity factors, N_c multiplied by the local shear strength. Most often a total stress approach is used for estimating penetration resistance.

Significant variations in the predicted penetration resistance can arise from (1) how one handles soil flow around internal stiffeners, particularly in respect of internal friction above ring stiffeners; (2) the choice of bearing capacity factors for internal ring stiffeners and external diameter changes; (3) the evaluation of interface friction. The first factor is the most significant. From the industry sponsored study, a consensus view is that the internal soil plug may not flow back around internal ring stiffeners for a significant plug height, and that when flow-back occurs it may trap an interface zone of water or high water content soft clay leading to low internal friction.

The selection of characteristic soil data was also identified as a key uncertainty, even in cases with reasonably good quality soil investigations.

Excessive underpressure may lead to the soil plug rising up within the caisson, without further penetration of the caisson. The critical underpressure is calculated from the inverse bearing capacity of the clay plug at skirt tip level, plus internal resistance from the inside skirt wall and inside stiffeners. Different approaches are possible in calculating the safety factor with respect to internal plug heave. However, since internal shaft resistance contributes both to the required underpressure for penetration and also internal plug stability, a suggested design approach is to evaluate external resistance using a material coefficient less than unity (representing worst case estimate of caisson resistance, and hence of required underpressure) together with a material coefficient greater than unity for the soil strength at the caisson tip (representing worst case estimate of plug base resistance). Maximum recommended penetration depth would then be determined as the depth where the material (safety) coefficient is around 1.5 (or 0.67 as appropriate).

Excessive plug heave may prevent full installation of a suction anchor. Provided soil plug failure is not approached, plug heave may be calculated on the basis of the soil displaced by some proportion of the caisson wall (between 50 and 100% assumed by the various predictors) and by internal stiffeners. The largest degree of uncertainty among the predictors was the assumed free-standing height of soil plug above internal ring stiffeners, and the extent to which water may be trapped between the soil plug and the caisson wall.

(2) Capacity

The capacities of suction anchors were predicted with plane limiting equilibrium methods, plastic limit analyses methods, and tailor made plane finite element methods. More approximate semi-empirical methods were not applied since they are not industry practice and are not recommended to be used for future important suction anchor applications.

The results of reference 3D finite element analyses from three organizations generally gave excellent agreement, and provided a good basis for checking the quality of the more simplified capacity prediction methods.

The comparison of results from simplified prediction methods and 3D finite element analyses showed that reverse end bearing can be calculated with the same bearing capacity factor, N_c, as for downward loading. The reverse end bearing seems best related to the average shear strength (average of compression, DSS and extension), and it is not recommended to weight the end bearing capacity towards the extension strength.

The plane limiting equilibrium method (optimal load attachment point) and the plane finite element analysis method (non-optimal load attachment point) where 3D effects were accounted for by side shear, generally gave good agreement with the 3D finite element analyses. The plastic limit analysis method using a function fitted to approximate upper bound results, Murff & Hamilton (1993), also gave good results. Available mechanisms that rigorously satisfied upper bound constraints indicated significant errors for shallow caissons, but gave good agreement for the longer caissons.

The capacity at intermediate load angles where there is coupling between vertical and horizontal failure mechanisms is well predicted when the interaction is determined by optimizing the failure mechanism in plane limiting equilibrium analyses. If the interaction is based on results from previous finite element analyses and model tests, one should be cautious if the conditions differ from those in previous analyses or model tests.

There is some uncertainty over conditions for a crack to form at the windward side of an anchor in lightly overconsolidated clay. In the case that a crack does occur, adjustments of the simple solutions to allow for a crack gave good agreement with finite element calculations undertaken using ABAQUS with zero tensile criterion on the total normal stress along the wall.

There is no industry consensus on the safety factor to use for capacity of suction anchor. The safety factor varies with regulatory agency and may also be client dependent. It should depend on the consequence of a failure.

Some factors that have been assumed and specified in the 3D finite element analyses, and that are thus not checked by the comparison exercise are (1) strain softening and resulting progressive failure, (2) set-up along the outside skirt wall, (3)

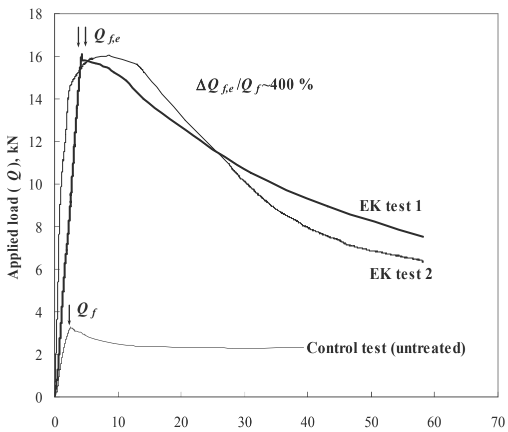


Figure 5. Increased pull-out resistance with electrokinetic treatment of clay (Micic and Lo, 2005)

interpretation of *in situ* and laboratory test data to establish a design shear strength profile, and (4) effect of large vertical displacements at failure.

Two papers in this session deal with anchor piles in clay: Sing et al and Micic and Lo. Micic and Lo present the strengthening effect of eletrokinetic treatment on a clay (Fig. 5). The beneficial effect is important. It would be worthwhile to study whether such techniques can be implemented in deepwater.

2.2.2 *Vertically loaded dragged anchors*

This section is quoted from the ISFOG keynote paper by Murff et al (2005). A vertically loaded anchor typically derives its holding capacity from a large bearing plate (called a fluke). During installation of a VLA, load is applied to the plate through an attached anchor line by various means such as through a connecting rigid bar (shank) or through a harness or bridle. The anchor is placed on the seafloor such that, as the anchor is pulled along the bottom, it penetrates the soil. Initially, the anchor dives more or less parallel to the fluke, eventually rotating such that the target penetration depth is achieved. Various methods are then used to 'activate the anchor', i.e. orient the anchor fluke so that it becomes perpendicular to the anchor line force. Often, the shank or bridle arrangement is designed to rotate after placement so that the anchor line load is approximately perpendicular to the fluke.

Conventional mooring lines are catenaries between the vessel and the seafloor such that the anchor line enters the soil horizontally. For deep water applications (> 300 m), this can result in the anchor being over 1000 m from the vessel. Owing to soil resistance, the anchor line takes on a reverse curvature below the mudline, such that the line imposes some vertical load component on the anchor, as illustrated in Figure 6. Historically, drag anchors have been used for temporary mooring systems although a few permanent installations. Their design has evolved by a systematic trial and error or iterative approach.

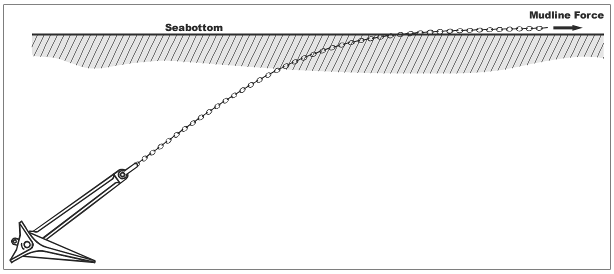


Figure 6. Schematic of drag anchor installation (Murff et al 2005)

Murff et al (2005) revised over 80 references relevant to VLA applications, including research reports from joint industry projects. There are three aspects of VLA behavior for which prediction methods are needed: (1) anchor line mechanics, (2) installation performance, and (3) holding capacity. Several prediction methods in each of these areas exist. Holding capacity prediction is usually implicit in the installation prediction methods. The converse is not necessarily true. Therefore, discussion of prediction methods for installation performance and holding capacity are often lumped together.

For a given embedment depth and orientation, as the load in the anchor line increases, the inclination of the line with the horizontal at the anchor attachment point decreases giving rise to an interaction between the anchor line and the installation/holding capacity of the anchor, the latter being dependent on the direction of the resultant force.

In general, the anchor line problem is approached in the same manner as that for predicting the displaced shape of a catenary, fixed at both ends, and deformed only by its own weight. In addition to the usual catenary forces, the soil exerts bearing pressure normal to the line and shear resistance tangent to the line. The governing differential equations for this system of forces are nonlinear and require an iterative, numerical solution such as the finite difference approach described by Vivatrat et al. (1982). Neubecker & Randolph (1996a, b) have proposed simplifying approximations (assuming small angles and weightless line) that linearize the equations and provide a surprisingly robust solution. The latter solution can be coupled with anchor behavior models to estimate both installation and holding capacity performance.

The capacity of a drag anchor depends strongly on its final orientation and depth below the seabed, hence prediction of the anchor trajectory during installation is a critical issue in VLA design. As discussed above, anchor capacity is only a special case of the installation sequence and, hence, the methods underlying installation prediction are directly applicable. Methods for predicting trajectory generally fall into four groups: empirical methods, limit equilibrium methods, plastic limit analysis methods, and advanced numerical methods.

Empirical prediction methods for installation are typically based on correlations with observed anchor performance (NCEL 1987, Vryhof Anchors 1999). These methods generally involve the prediction of anchor depth and capacity as a function of the anchor weight and, at least, a crude measure of soil strength.

Limit equilibrium methods generally take into account a more detailed description of the soil and the anchor (Stewart Technology Associates 1995, Neubecker & Randolph 1996b, Dahlberg 1998). The anchor line mechanics are combined with the model, at least in a simplified way. Limit equilibrium methods are typically incremental methods based on an estimated distribution of soil forces on the anchor at its failure condition

Plastic limit analysis is in many ways similar to limit equilibrium methods but also fundamentally different. A solution is determined using an assumed failure mechanism along with virtual work principles. The calculated failure load is minimized with respect to the geometric parameters defining the mechanism. The only method that we identified in this category is that of Bransby & O'Neill (1999). This is a simple, elegant idea and achieves results similar to those that the limit equilibrium methods purport to achieve, except that the force distributions on the anchor are implicit in the assumed mechanism.

Advanced numerical methods (usually the finite element method) have the potential to obtain a rigorous solution for all aspects of anchor behavior. In practice, however, they have considerable practical limitations because a simple anchor trajectory prediction would require a prodigious effort. On the other hand, numerical analysis can be used to advantage to check calculations for a specific snapshot in the anchor trajectory and to therefore enhance the other prediction methods described above, as shown by Rowe & Davis (1982) and Bransby & O'Neill (1999). The Murff et al (2005) study concluded the following:

An hypothetical case study was carried out to assess the variability in predictions of drag anchor installation performance for currently available numerical models. These models typically employ limit equilibrium methods using simplifying assumptions. All models predicted qualitatively similar trajectories with the anchor initially penetrating parallel to the fluke. The anchor gradually rotates and ultimately reaches a steady state where the anchor translates horizontally. The most important effects were anchor line diameter, fluke-shank angle, fluke aspect ratio, and soil strength characteristics. For the base case, the average simulation depth prediction (assuming a wire rope forerunner) was approximately 2.5 times the conventional chart prediction (assuming a chain forerunner and correcting for differences in drag distance) and the average mudline ultimate capacity was approximately 3.5 times the conventional chart prediction. Some of this difference is due to the larger depths and capacities associated with wire rope vs. chain. The predicted ratio of uplift capacity (load normal to fluke) to installation load ranges from 1.5 (simulations) to 2.0 (charts).

To provide further insight into simplified industry anchor installation models, a back-calculation was done using four different sets of drag anchor or plate anchor installation data, including measured anchor response. These data represented some of the best test data available but was not ideal in several respects. Each calculation used a different model with its own set of assumptions and methodology. With sufficient 'fine tuning' of the input data, all models were capable of reasonably replicating the installation trajectories and loads for the majority of the tests. The predicted trajectories of the more conventional drag anchors were curved upward and generally in good qualitative agreement with the measurements. For the two plate anchors, the measured data showed a much more linear variation of depth with drag distance whereas the predictions were again curved upward. It was observed that mudline load was a very linear function of penetration depth in all cases except possibly in close proximity to the mudline. Likewise the models all seem to predict this trend. This emphasizes a very important point: the dependence of penetration and load on drag distance seems to introduce the largest uncertainty. An additional study was carried out using conventional chart prediction methods, similar to the NCEL charts (1985) included in API RP2SK (1995), to back-calculate the four field tests. The chart predictions provided reasonable estimates of the field tests for drag distance, depth, and mudline load except for Case 4 which had significantly different anchor line and fluke-shank angle than those on which the chart prediction was based. This agreement is contrary to the significant differences between chart predictions and simulation predictions for the hypothetical cases.

To supplement the experimental results on plate anchor capacity and to develop a better understanding of the behavior of plate anchors under multi-axial loading, a series of finite element calculations was conducted. Failure surface diagrams were developed for multi-axial loading among normal, parallel and moment loads. Coupling is strongest between normal load and moment. At high normal loads the anchor undergoes significant rotation at small moments. The FEM results were in good agreement with available analytical results with a tendency to slightly observed values. The largest discrepancy was found to be in the parallel capacity. The FEM results were also used to gain insight into the simplified methods used to estimate anchor trajectories. The FEM results were compared to the inferred load capacities of selected 'snapshots' from the trajectories predicted in the hypothetical studies. These comparisons point out that prediction of anchor trajectories is a difficult task and the simplified methods should be calibrated for specific applications in order to provide the most reliable results.

2.3 *Offshore geohazards assessment*

Working at greater and greater water depths has been the focus over the last years. Although the technology for drilling and production developed separately, their evolutions follow similar curves. In less than 20 years, deepwater milestones in the Gulf of Mexico have gone from depths of 300 m to 2000 m. Three papers in this session touch the subject of geohazards: Strout and Sparrevik on the pore pressure in the sediments and their measurement, Puech et al on the underwater slopes offshore West Africa, and Kliner and Grozic on gas hydrates. Two other papers in this session (Cheuk et al, and Vanden Berghe et al) deal with offshore pipelines, for which slope stability and other geohazards are important design issues.

Geological risks, or "geohazards", are events due to geological features and processes that present severe threats to humans, property and the natural and built environment. Various geological processes, earthquakes and human activities, for instance in connection with petroleum exploration and production, can trigger slides and large mass flows. Geohazards are a major issue in deep water, mainly because the nature, extent and effects of geohazards are not well known offshore. Geohazards include for example, submarine slides, gas hydrates and free gas, over-pressured sand zones, and very soft, brittle soils such as oozes, earthquakes, diapirism, gas leakage, fracture zones. These need to be carefully evaluated before field development can start.

Fig. 7 and 8 illustrate some of the offshore geohazards, and their consequences. An important issue is the detrimental effect they can have on the capacity of a foundation or anchoring system. The integrated evaluation by geologists, geophysicists and geotechnicians is essential.

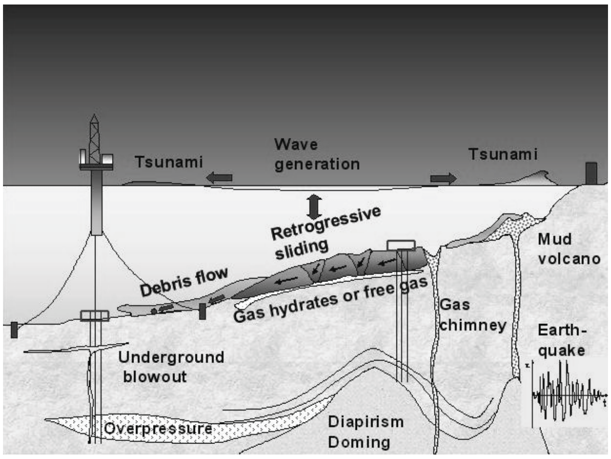


Figure 7. Offshore geohazards (Kvalstad personal communication 2002)

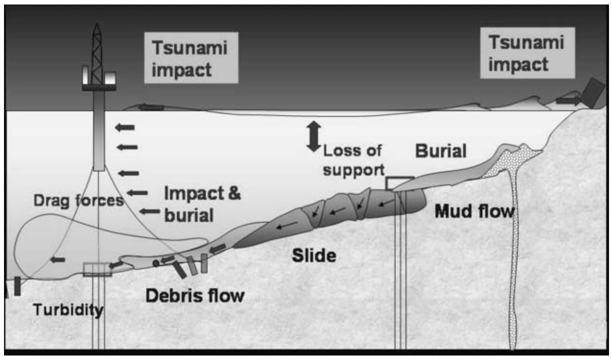


Figure 8. Impact of offshore geohazards on structures at sea and on land (Kvalstad personal communication 2002)

The triggers for different failures can be natural on-going geological processes or human activities. A distinction can be made between stress (or load) increasing triggers bringing the stress conditions in the soil mass closer to failure, and strength decreasing triggers causing strength loss due to large strains and pore pressure changes. Triggers include for offshore slopes (Nadim, 2002):

- human activities;
- earthquake activity causing short-term inertia forces and post-earthquake pore pressure increase and fault displacements;
- rapid deposition leading to excess pore pressure conditions, underconsolidation and increased shear stress level in a slope;
- toe erosion or top deposition giving higher slope inclination and increased gravity forces and shear stress along potential failure surfaces;
- sensitive (contractive) and collapsible soils, which could lead to retrogressive sliding and increased spatial extent of failure zones;

- melting of gas hydrate in a deepwater environment caused by temperature increase or pressure reduction leading to increased pore pressure and reduced soil strength;
- active fluid /gas flow and expulsion;
- mud volcano eruptions and diapirism giving rise to mass wasting and soil displacements;
- sea level lowering during glacial periods leading to lower pressure, free gas expansion and gas hydrate melting (relevant for evaluation of ancient slide activity);
- increased seawater temperature at sea bed level caused by changes in current regime leading to temperature increase in the soil mass and melting of hydrates (relevant for evaluation of ancient slide activity).

An example of this overwhelming new direction in the offshore profession is the number of geohazards studies and the several papers on the topic of geohazards in deep water at the SUT and OTC conferences these recent years.

Predicting the hazard posed by geological processes, and evaluating the human, environmental and economical consequences of geohazards require that each of the uncertainties in the situation to analyse are quantified and addressed properly.

A state of the art study was made for the extremely large underwater Storegga slide where the offshore Ormen Lange gas field is located. The results have been compiled by Solheim et al. (2005) in the thematic volume of Marine and Petroleum Geology, soon to be issued as a textbook.

The Ormen Lange gas field is located at water depths of about 800 to 1100 m. The field is situated approximately 120 km from the coastline, within the scar of the gigantic prehistoric Storegga Slide. The slide, that took place about 8.200 years ago, is one of the largest known submarine slides in the world.

The Ormen Lange gas field was discovered in 1997 and is the second largest gas reservoir in Norway. The field is not only within the scar from the Storegga slide. It is also close to the steep headwall of the slide. This gigantic submarine slide occurred 8200 years ago. The headwall of the slide is about 300 km long and the total run-out distance was close to 800 km. The estimated slide volume was 3.000 km^3. The slide caused large waves (tsunamis) that also reached the coasts of Norway, Scotland, Shetland and the Faeroes. The heights of the tsunamis were up to 10-12 m along the coast of western Norway and locally up to about 25 m on Shetland (Bondevik et al, 2003).

A regional geological model was required to explain the Storegga Slide (Bryn *et al*, 2003; 2005. Kvalstad, 2002). The slide was most likely initiated 200 km downslope from the Ormen Lange gas field and developed as a retrogressive slide. The unstable sediments in the area disappeared with the slide 8.200 years ago. A new ice age with infilling of glacial sediments on top of marine clays in the slide scar is needed to create a new unstable situation. The slopes around the Ormen Lange field are stable and the slope stability will not change due to the Ormen Lange activities (Bryn et al, 2005; Kvalsqad et al 2005 a, b). The positive conclusion of this work made it possible to approve development of the field.

In the thematic volume of Marine and Petroleum Geology, the following aspects are considered: present day conditions, geological development, seismicity, slide dynamics and tsunamis, slide triggers, factors affecting present slope stability, and a risk assessment for future exploitation of the field. The volume is strongly recommended for those interested in underwater slope stability.

3 TRANSPORTATION GEOTECHNICAL ENGINEERING

Increase in life-time and serviceability of transport infrastructures (roads, airports and railways) is essential to reduce maintenance and consequently save costs and contribute for the quality of life.

A major concern affecting these aspects is the assessment of quality of materials used in the structure, including subgrade soils, as well as the quality control during and after construction. This of course assuming a proper design is done. These topics are developed hereafter.

3.1 *Some peculiarities of materials behaviour and environment aspects*

It is well established the non-linearity behaviour of soils and unbound granular materials due to strain and stress levels. Consequently, it is necessary to specify what type of modulus we are dealing with, as illustrated in Figure 9. This is a major concern for characterization, design and construction control.

Soils and unbound granular materials in pavements and rail tracks are submitted to complex stress field involving a very large number of cycles, in general several millions for the service life of these structures. Moreover, subgrade soils of these structures can be in situ soils or compacted soils used as construction materials.

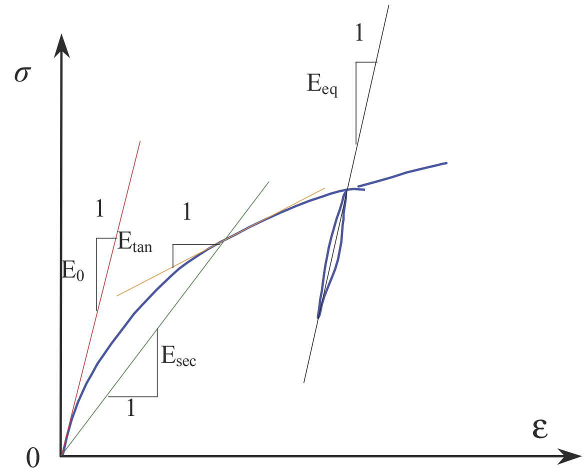


Figure 9. Modulus in a non linear stress-strain material behaviour

The mechanical properties of these soils are in many cases improved by stabilization or by reinforcement to respond to the performances required for these transportation structures at the level of the earth structure or even at the level of the pavement or rail track structure as a capping layer (Wilkinson et al; Gnanendran; Brandl, 2001a) . This field of soil mechanics dealing with construction materials satisfying physical, hydraulic and mechanical properties is rather limited for natural soils where engineering properties need to be measured, rather than being specified (Worth and Houlsby, 1985). In particular, for roads, airports and railways, embankments where non saturation conditions should be maintained during the service life of the structure, the theories for non-saturated soils should be applied. This aspect is important since the performance of embankments should be verified not only for the ultimate state, but particularly for the serviceability state. This last criterion is essential in design of embankments on compressive soils and the contribution of Ed Calle et al. is relevant for improving accuracy in settlements predictions.

Creep is also an important phenomenon of soil behaviour that must be carefully evaluated in order to a better prediction of long term-settlements and to avoid problems in the built structures (see Figure 10 from Akai and Tanaka, 1999).

Unbound granular materials, generally crushed aggregates, are used as constructions materials for the structural layers in pavements (sub-base and base courses) and rail tracks (sub ballast and ballast layers). Due to environmental concerns, these natural materials should be replaced by processed materials coming from recycled municipal wastes, demolishing building materials and by-products from industry.

The environmental pressure to use new (non traditional) materials in transportation construction, as well as the need to deal with more severe traffic conditions change the traditional empirical approaches of material characterization and design of pavements and railways to more mechanistic approaches (performance based), where geotechnical background becomes of relevance. This is also been changing in education where Professors with Geotechnical background become more and more involved with lecturing and research on transportation issues. It is therefore a challenge to the Soil Mechanics Society and the other sister Societies to be more concerned than before with the transportation field which has a big impact in the economy of a country and the quality of life.

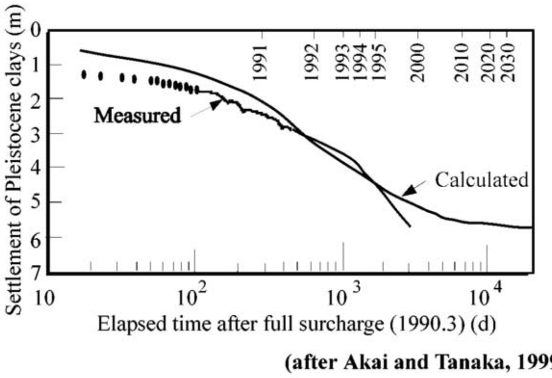


Figure 10. Analytical and measured settlement in a Pleistocene clay (Akai and Tanaka, 1999)

3.2 *Assessment of materials by laboratory tests*

A summary of the framework of the mechanical soil behaviour that must be taken into account in characterization, design (material model) and construction (mechanical evaluation) of pavements has been done by Brown (1996) and more recently by Gomes Correia (2004). Attention was addressed to the non-linear behaviour of soils and unbound granular materials, as well as to the interpretation of test results using the effective stress concept for unsaturated soils. It summarises the effort put by the scientific and technical community in recent years to develop test protocols, modelling and analysis techniques suitable to better assess mechanical properties and performance of compacted soils and unbound granular materials. Some of these highlights will be evoked here, complemented with some other aspects more relevant for railways. This will help the discussion related with some of the papers in this session.

The routine laboratory characterization of the soils and unbound granular materials involve generally index tests and mechanical tests (triaxial and oedometer). In the case of unbound granular materials routine tests are even more oriented to the intrinsic particle characteristics than to the full grain size material characterization.

Most of actual standards assess material quality based in these properties. For instance for soils, the properties generally specified to reuse a soil in earthworks and capping layers are: grain size distribution, in particular the percentage of fines, the sand equivalent, the methylene blue, liquidity limit, the plastic index, maximum density related with Proctor (standard or modified) and CBR. It is obvious that these criteria based on empirical relationships cannot optimise material use nor promote the use of new materials. However, it should be noticed that these index properties are very useful. In the framework of soil mechanics a big effort has been done to estimate parameters of complex elastoplastic soil models (Hujeux, 1985) based on correlations with the grain nature (identification parameters) and packing of grains (relative density or plasticity index), as reported in the paper of Elmi and Favre.

For unbound granular materials, the tests consider particle characteristics, partial fractions of full material and only few tests are concerned with the full grading material. The current tests are generally to evaluate the following properties: particle shape, size and density, cleanliness, percentage of crushed particles, resistance to fragmentation/crushing, volume stability, water absorption/suction, composition/content, resistance to attrition, dangerous substances, durability. Illustrations of results of some of these properties are presented in Wijeyakulasurija et al.

The road specifications to ranking unbound granular materials are generally based in empirical tests based only in a partial fraction of the full material, in occurrence Los Angeles and Micro-Deval (Paute et al., 1994). It must be noticed that recent studies carried out during a research project (Courage, 2002 – see http://www.civeng.nottingham.ac.uk/courage) confirms previous research results (Paute et al., 1994; Petkovsek and Gomes Correia, 2002), demonstrating that classifications based in index tests, some even based in particle characterization or grading fractions, are not reliable to predict the mechanical behaviour of the full grading material associated with specific state conditions. This is noticeably more important when dealing with new materials, where empirical classifications can not be applied. Wijeyakulasuriya et al. corroborate these findings.

An change in the conventional characterization of subgrade soils and geomaterials for pavements and railways is necessary, as well as in the material specifications is urgently needed. The characterization should be done using performance based tests reproducing stress and strain induced by traffic and testing the full grading distribution curve of the material in the state conditions prevailing during the service life of structure. These kinds of tests are still mostly used in research, but it is definitively desirable to turn them to more practical use. They can satisfy the purpose to rank materials based in a mechanical approach and to obtain parameters for material constitutive law to be used in mechanistic design.

Related with these tests, is necessary to clarify the terminology. Monotonic or cyclic loading tests are performed at different load or strain rates in which the effects of inertia on the stress field of the material are negligible. The evaluation of the stress-strain behaviour of materials under these tests involves stress-strain measurements or load-displacements measurements. In contrary, dynamic loading tests generate inertia forces which should be considered in analysis, generally done using equation of motion. In this case time histories of particles velocity or acceleration are measured.

3.2.1 *Triaxial cyclic loading test*

In pavements, the most used laboratory research tool for mechanical characterization is the triaxial cyclic loading test (Gomes Correia et al.; Wijeyakulasurija et al.; Erlingsson). It is in addition used in railways applications, as well as is dynamic test. Less used are the ring shear test (Vallejo and Chik) and the cyclic simple shear (Shaw and Brown, 1986). A more fundamental laboratory research tool is the hollow cylinder (Chan and Brown, 1994). These equipments should be able to measure small to intermediate strains, which are representative in soil subgrade and unbound pavement layers during the service life (Gomes Correia and Biarez, 1999, Gomes Correia, 2001, 2004).

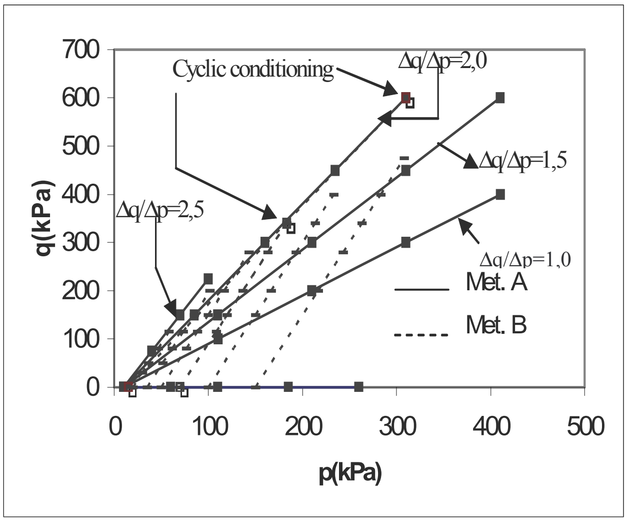
The cyclic triaxial test is standardized by AASHTO standard (AASHTO T 294-92 I) and by CEN standard (EN 13286-7, 2004). This last standard is very recent and more complete than AASHTO which is more restrictive in stress paths and adopting different range of stress levels. In fact the test protocol of CEN standard is appropriated to obtain parameters for modeling quasi-elastic and plastic behaviour, as well for ranking materials, incorporating an informative annex with guidelines for these purposes. The main issues of this standard have been reported by Gomes Correia (2004) and results of its use are presented by Wijeyakulasurija et al.

The cyclic load triaxial test consists of imposing on a specimen, under specific state conditions (density and water content/suction), cyclic stresses and in measuring the axial and radial strains and eventually pore water pressure. In EN 13286-7 three test procedures are described to determine mechanical properties of UGM that can be used for performance ranking of materials and for analytical and numerical pavement design procedures. To simulate the state conditions and also evaluate the sensitivity of material to water content and density, several representative field state conditions are proposed.

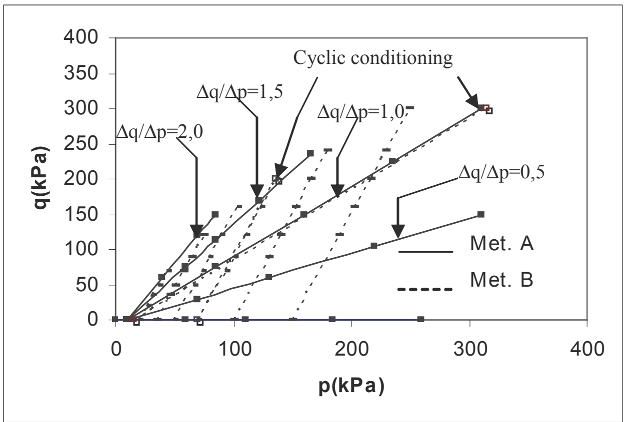
To simulate the stress states induced by the moving loads two methods are proposed:

Method A, designed *Variable Confining Pressure (VCP)*, where a cyclic axial deviator stress and a variable (cyclic) confining cell pressure, varying in phase, are applied (fig. 11).

Method B, designed *Constant Confining Pressure (CCP)*, where a cyclic axial deviator stress and a constant confining cell pressure are applied; it is a simplified stress regime requiring a more simple equipment (fig. 11).



a) HSL – High stress level



b) LSL – Low stress level

Figure 11. Stress paths in EN 13286-7 test protocol: a) HSL – high stress level; b) LSL – Low stress level

These two methods are used to obtain the stress-strain behaviour under unload-reload cycles of quasi-elastic behaviour (close loops) as shown in Figure 12. For this purposes the test procedures start with a cyclic conditioning, (several thousand cycles at a high level of stress - 20000), applied to stabilise the permanent strains. The consequence of this pre-loading is to change the elastic threshold of the material that seems to happen during the construction phase of the pavement. Then, a series of short loadings (about 100 cycles each), at different stress levels (fig. 11), less severe than the cyclic conditioning, are applied. These series of short loadings allow determination of the quasi-elastic parameters of the material model, which can be more or less complex. This test procedure has proved very suitable to obtain parameters for the Boyce model (Boyce, 1980; Gomes Correia, 1985). Wijeyakulasurija et al. test different unbound road base materials showing that the quasi elastic behaviour is almost constant with the number of cycles, except for one material where an increase of unload-reload modulus is observed (fig. 13). This could probably be explained by the material crushing with cyclic loading.

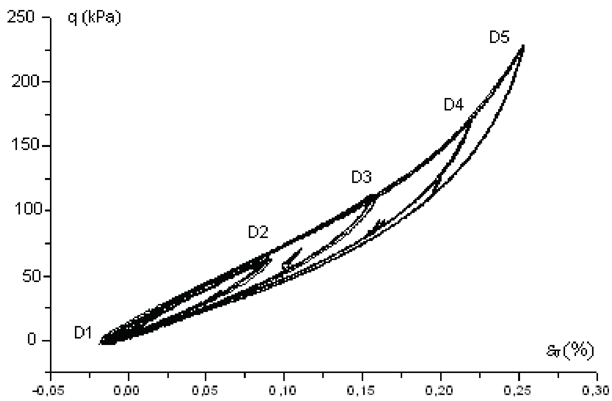


Figure 12. Closed loops of a granular material (granite aggregate 0/31.5mm) under unload-reload cycles after a preloading of 21.000 cycles with a constant deviator stress amplitude of 230 kPa and under a confining pressure of 40 kPa (Gomes Correia et al. 2005)

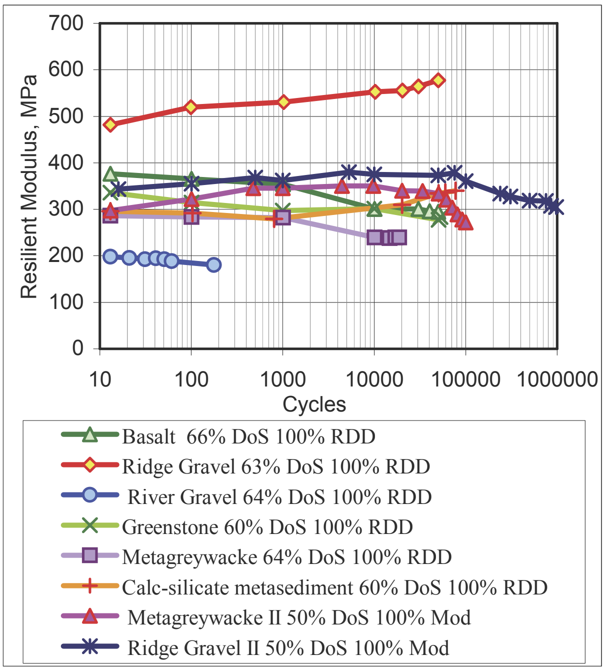


Figure 13. Unload-reload modulus as a function of number of cycles (Wijeyakulasurija et al. 2005)

Two different series of stress paths levels are proposed in the EN 13286-7 (2004) standard to accommodate representative stresses to which the material will be submitted in field: low stress levels (LSL) and high stress levels (HSL). Other test procedures are also proposed in this standard to study the permanent deformations of the UGM. For a rapid evaluation of permanent deformations produced by different stress levels, a so called multi-stage procedure is also described.

In the perspective of a routine test, the EN 13286-7 propose two cyclic load test procedures: a single stage procedure (SSP) where only one stress path is repeatedly applied on each specimen and a multi-stage procedure (MSP) where several different stress paths are applied successively on the same specimen.

In both procedures, the tests are carried out generally by applying a large number of load cycles with constant stress amplitude and measuring the increase of the permanent strains with increasing number of load cycles. For the SSP at least 80 000 cycles are applied for each stress path and for the MSP 10 000 cycles are applied for each stress level.

3.2.2 *Material behaviour and performance under triaxial cyclic loading tests*

Very dense compacted unbound granular materials (void ratio around 0.2), as required for pavements (capping layer, sub-base and base courses) and rail track (capping layer and sub-ballast layer) that have been subjected to a cyclic preloading history representative of construction traffic loads, exhibit a modulus versus strain curve typically with a shape as presented in Figure 6 (Gomes Correia, 2004; Gomes Correia et al.). This behavioural trend is essentially due to an increase in the vertical Young's modulus, E_v, associated with an increase in σ_v, and can continue until the dilatant behaviour, due to relative rotation of particles with slipping, becomes significant. This trend of stiffness corresponds to an S-shaped stress-strain relation when subjected to unloading and reloading with a relatively large stress amplitude (fig. 14). Subsequently, the material starts showing a significant reduction in the stiffness with an increase in the strain as a consequence of an increase of irreversible (or inelastic) strain at an increasing rate with an increase in the strain. This result shows that, to obtain a normalised decay relation (i.e., an $E_{tan}/E_{max} - \log(\varepsilon_v)$ curve) representing the non-linearity only by strain increase, it is not relevant to normalise the respective measured E_{tan} value with the initial value of E_v (when at ε_v= 0), E_{max}, as reported by several other authors. Indeed, the E_v value at ε_v=0 is not the maximum value for the E_{tan} values, but is the minimum value of the E_v values for the subsequent triaxial compression loading history (Gomes Correia et al.).

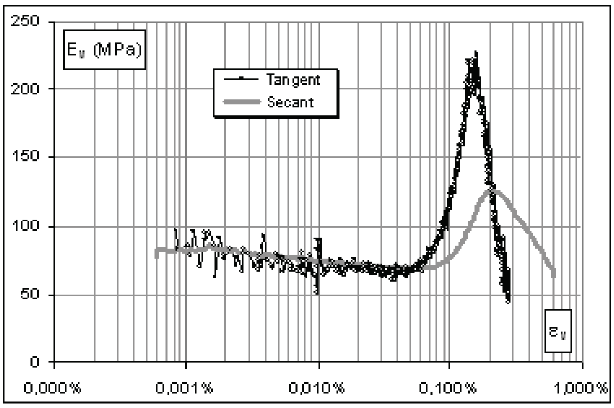


Figure 14. Tangent and secant moduli as a function of strain level for a very dense compacted granite 0/31.5 (Gomes Correia, 2004)

This trend of behaviour has been observed also with cyclic pre-strained Toyoura and Hostun sands and very dense Chiba gravel (Tatsuoka et al. 1995, 1999), as shown in Figure 15.

Based in the results of triaxial cyclic loading tests, the ranking of mechanical behaviour of unbound granular materials was originally proposed in France for base layers of low traffic pavements (Paute et al., 1994), as already mentioned in Section 3.2. This kind of classification of materials uses the two main mechanical properties responsible for the material performance in the pavement layer or in the sub-ballast layer of a rail track, the stiffness and resistance to permanent deformation. From the cyclic triaxial loading test, these two parameters are defined as a characteristic value of secant modulus, E_c, determined for the cyclic stress values of Δp=250 kPa and Δq=500 kPa and a characteristic permanent axial strain, ε_1^p, obtained during cyclic conditioning and calculated as follow:

$$\varepsilon_1^c = \varepsilon_1^p(20000) - \varepsilon_1^p(100) \qquad (1)$$

where: $\varepsilon_1^p(20000)$ is the permanent axial strain at the end of the cyclic conditioning (20000 cycles) and $\varepsilon_1^p(100)$ is the axial strain at the end of the first 100 cycles. These values are obtained for the material compacted in the following state conditions: water content, w, equal to: $w_{OPM} - 2\%$; w_{OPM} being the optimum water content of modified Proctor; dry density, ρ_d, equal to: $0{,}97 \times \rho_{dOPM}$; ρ_{dOPM} being the maximum dry density of modified Proctor.

Figure 16 shows the boundaries proposed by EN 13286-7 for three classes of mechanical performance C_1 to C_3.

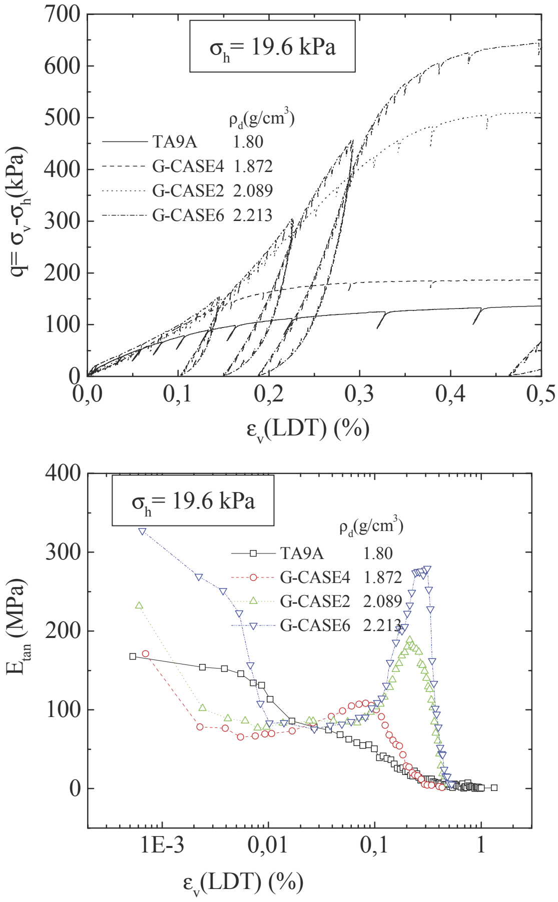


Figure 15. Relationships between (a) the deviator stress and the vertical strain – S shape; (b) tangent modulus and vertical strain, obtained from triaxial compression tests on moist Chiba gravel (Jiang et al, 1998 and Jiang and Kohata, 1998, cited by Tatsuoka et al., 1999)

Another alternative is ranking material on the basis of only their resistance to permanent deformations. In fact, permanent deformations lead to rutting at the surface of the pavement and, hence, are of great importance with respect to riding comfort and safety. For this purposes three ranges of material deformation behaviour are considered: (1) plastic shakedown – range A; (2) plastic creep – range B and (3) incremental collapse – range C. Figure 17 illustrate these different deformation behaviours.

An application of this approach is presented by Wijeyakulasurija et al. and illustrated in Figure 18. It is noticeable, as mentioned by the authors, that Ridge gravel is in range A contrarily to river gravel that is in range C. Based on these results,

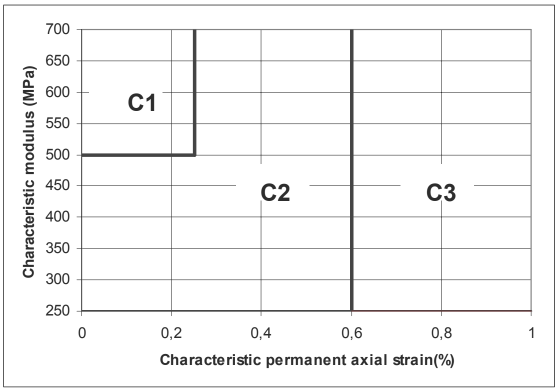


Figure 16. Classification for ranking mechanical behaviour of UGM (EN 13286-7)

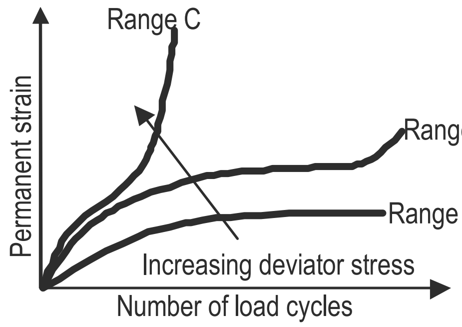


Figure 17. Different ranges of material deformation behaviour

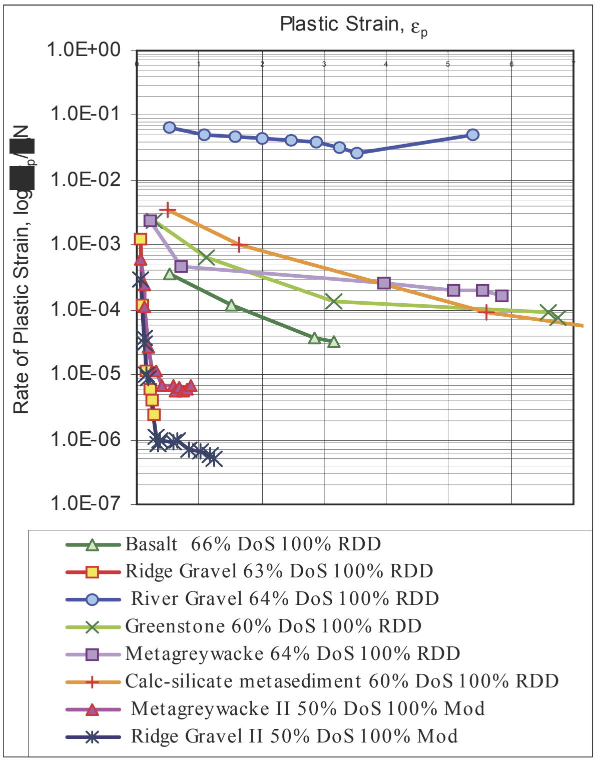


Figure 18. Rate of change of plastic strain vs plastic strain (Wijeyakulasurija et al. 2005)

the authors conclude with a tremendous divergence of the crushed rock material performances when compared with the application of the conventional national material specifications.

As mentioned in Section 3.2, these results corroborate previous findings revealing that empirical specifications used worldwide may not contribute to the best optimization of material applications. However, this should be confirmed with full scale experiments to validate the laboratory results. In this context, the research of Wijeyakulasurija et al. is a first contribution showing the good agreement of laboratory material assessment by cyclic triaxial tests with field observations for two materials.

It is noticeable that triaxial cyclic loading tests did not reproduce the rotation of principal stresses occurring due a moving wheel. This has important effects in permanent deformations as demonstrated by Wijeyakulasurija et al. comparing the results of the cyclic triaxial tests with results of a physical model using a moving wheel with bi-directional travel.

3.2.3 *Dynamic loading test*

In the laboratory, dynamic loading tests, such as resonant column (RC) or wave propagation tests, are used to obtain stress-strain behaviour of geomaterials at strain levels less than 0.01% or 0.001%. However, nowadays these strain levels and also strains exceeding these limits are also possible by laboratory static stress-strain tests using a single specimen.

It is also well established that static and dynamic tests, performed under the same conditions, give essentially the same quasi-elastic properties for many types of soils and soft rocks (Jamiolkowski et al., 1994; Porovic and Jardine, 1994; Tatsuoka et al., 1997; Tatsuoka et al., 1999). However, for gravels and other unbound granular materials with large size particles, this conclusion is not confirmed. In fact, Modoni et al. (1998), Anh Dan et al. (2002) obtained very different values from pulse wave transmission and static cyclic tests in a gravel with D_{max}=38mm. Figure 19 shows a ratio of 2.2 between the elastic Young's modulus obtained from the velocity in the wave transmission tests ($E_{v(d)}$) and the value of ($E_{v(s)}$) obtained from cyclic tests, both under the same stress states (Modoni et al., 1998). The effect of the ratio of the body wave length to the size of particle size for unbound granular materials (gravel and ballasts) is still a matter of discussion and needs further research. This aspect is of relevance when analyzing dynamic tests in the field using wave propagation. The main question is where is the boundary between a continuous medium and a particulate medium.

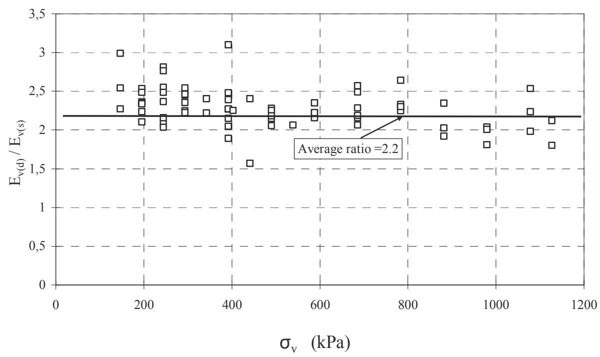


Figure 19. Ratio of the elastic Young's modulus obtained from the velocity in the wave transmission tests ($E_{v(d)}$) to the value of ($E_{v(s)}$) obtained from cyclic tests (Modoni et al., 1998)

Another important aspect dealing with dynamic aspects is the damping ratio of the material. This parameter is very sensitive to strain rate, but is also influenced by the number of cycles. In fact, under an accommodation state of the materials, the loops became more and more closed with increasing number of cycles. Figure 20 illustrates this type of behaviour which should be taken into account for long term material performance (Tatsuoka et al., 1995). This aspect may have some influence in rail track performance.

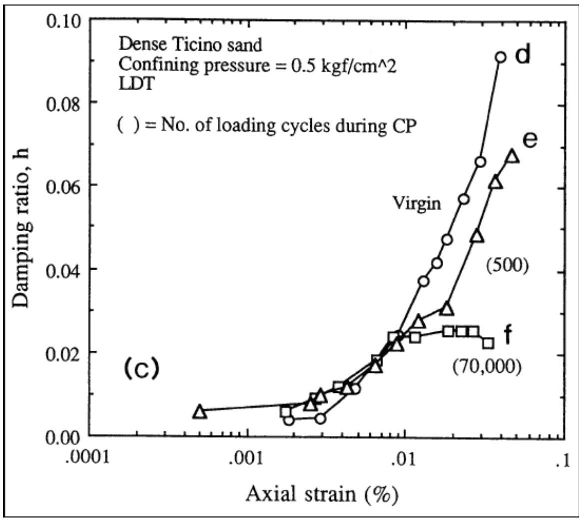


Figure 20. Influence of number of cycles in damping ratio of sands (Tatsuoka et al. (1995)

3.2.4 *Assessment of materials by physical model tests*

Other performance based tests are the physical model tests, using a rolling wheel (Wijeyakulasurija et al.; Momoya and Sekine; Hayano and Kitazume; Erlingsson) or a no moving cyclic loading system, like a plate or wheel submitted to cyclic loading, generally vertically (Queiroz and Macari; Abdelkrim et al.), but also horizontally, as reported by Gnanendran. Only exceptionally, shaking table (Ishikawa and Sekine, 2003) and centrifuge tests are also used (Vinogradov et al.).

Besides the requirements of similitude laws necessary to design these physical models, it is very important to take into account the boundary conditions, mainly when dynamic loadings are applied. In general, most of the physical models involve quasi-static loads where inertia forces may be negligible.

These physical models are used many times to rank materials and evaluate conceptual solutions. The former is illustrated by Wijeyakulasurija et al. and the last aspect is well exemplified by Gnanendran in an airfield application and by Queiroz and Macari and by Momoya and Sekine in a railway applications.

They are also frequently used to validate/calibrate laboratory test results and constitutive laws (Abdelkrim et al.; Erlingsson; Hayano and Kitazune).

These physical models should then reproduce in a small scale the real field behaviour. In this framework, fixed-point loading tests, using a cyclic loading plate test or cyclic wheel loading test, have been conducted, as reported by Abdelkrim et al., and by Queiroz & Macari for railways applications. However, as stated by Momoya and Sekine, these tests are of limited application since they do not reproduce correctly the field behaviour of a rail track. The main drawbacks are: (1) maximum load applied to the sleeper beneath the fixed loading point gradually decreases with repeated loading and consequently cannot evaluate resilient deformation characteristic of rail track correctly; (2) the continuous rotation of the direction of principal stresses cannot be simulated as it happens under moving wheel loading, affecting the permanent (or residual) deformations results (Wijeyakulasurija et al). These drawbacks were overcome by the physical model developed by Momoya and Sekine mounting a moving-wheel loading system applying a constant load in the rail, simulating the traveling trainload on rail track. Figure 21 shows results obtained with this loading system on two different rail track structures, one with only ballast over the subgrade soil and another with an asphalt roadbed intercalated between ballast and subgrade soil. These results show very clearly the decrease of strain rate when repeated loads are applied in a fixed-point in comparison with the moving loading, besides the better performance of the structure with the asphalt roadbed (Momoya and Sekine). These authors also report that the quasi-elastic strains keep constant during the number of cycles where important developments of permanent deformations were observed, corroborating the same findings of Wijeyakulasurija et al.

It is recommended that these physical model test results be also validated by full scale field experiments and more correlation with laboratory tests be made. In this context, the monitoring system to measure the stress distribution of vertical and shear stresses in materials developed by Hayano and Kitazune should be of great importance.

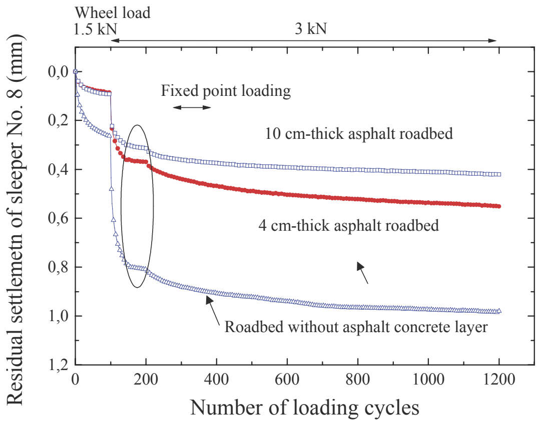


Figure 21. Permanent settlements of a sleeper (Momoya and Sekine, 2005)

3.3 *Field assessment of quality construction control*

An inventory of the in-situ assessment of subgrade materials was carried out and reported by the PIARC technical committee (C12) on earthworks, drainage and subgrade (PIARC, 1995). Specifically for unbound granular materials, an inventory of the in-situ mechanical tests used at the European level was also realized during the COST 337 action (COST 337, 2002: www.cordis.lu/cost-transport/src/cost-337.htm). From these two surveys it can be concluded the following (Gomes Correia, 2004):

(1) To understand the role of in situ tests, it is necessary to look at the role of design methods; in this context index tests (e.g. density, water content, CBR) are oriented towards empirical design methods, while sound mechanical tests are more suitable to mechanistic design methods.
(2) The in situ CBR test is still used in several countries, despite its empirical character and its questionable relationship with material performance. Index tests, like CBR and dynamic penetration tests (DPT) or dynamic cone penetrometers (DCP), are generally used as controls or ranking tools, being their results related to empirical knowledge.
(3) The likely performance of the subgrade soils and UGM should be assessed by those tests that simulate the traffic loading and from which load deformation criteria can be obtained (stiffness).
(4) The two mechanical performance tests used almost everywhere is the static plate bearing test (SPLT) and the Falling Weight Deflectometer (FWD).
(5) Other common measurements in situ methods are: Ménard Pressuremeter, dynamic plate loading tests, French dynaplaque and Lacroix deflectograph, instrumented compaction rollers (continuous compaction control), continuous bearing capacity meters, geo-radar and spectral analyses of surface waves (SASW).

(6) The use of dynamic plate loading tests is increasing, because they are simpler and faster than static plate bearing tests. Various types of equipment are available for these tests; they differ by their size, measurement principle and by the parameters determined (deflection, stiffness, etc.).

Two papers in this session deal with field mechanical performance tests, one oriented to construction quality control of earthworks (Edil and Sawangsuriya) and the other (Lee and Stokoe) oriented for project level. Both are dynamic non-destructive devices. The former is so-called soil stiffness gauge (SSG) and is a local (spot) test, contrary to the other that is so-called rolling dynamic deflectometer (RDD), that is a continuous measurement device.

With respect to the mechanical performance tests for construction quality control, it must be encouraged to move from the local measurements to the continuous measurements where theoretical investigations and field observations are now available and put in practical applications (Thurner (1999), Adam (1999), Quibel, 1999; Adam and Kopf, 2000; Krober et al., 2001; Brandl, 2001a). As referred in Brandl (2001a), "the hitherto used conventional methods of soil mechanics have proved suitable for decades but are based only on spot checking by more or less random or subjective selection or along grids". Continuous measurements of stiffness or deflections become important where high uniformity and homogeneity is required, as it should be the case for embankments of roads, airports and particularly railways, as well as for the structural layers of their structures. In this context, three non-destructive methods are available: roller-integrated continuous compaction control (RICCC), "Portancemeter" and spectral analysis surface wave method.

RICCC possesses the essential advantage that continuous compaction optimisation is done during the compaction process, since the roller is the measuring tool. Moreover, recent innovations automatically interact with roller response (intelligent compaction), thus optimising compaction (Adam and Kopf, 2000) and consequently causing less grain crushing and attrition which benefits in the mechanical properties of the materials (Vallejo and Chik). In contrast, the other two techniques require separate external test equipment, where the roller itself is the measurement tool, and therefore are post-compaction control.

The Portancemeter was developed in France. It consists of a wheel (1 m diameter, 200 mm wide), equipped with a vibratory loading system (like a roller) and instrumented with accelerometers. The system allows continuous determination of the stiffness of the evaluated layer, at a speed of 3 km/h (depth of action about 600 mm). The wheel is mounted on a trailer that can be towed by a vehicle. This equipment has the advantage, in relation to RICCC, of applying the load actions under normalized conditions (same boundary conditions) and consequently is more appropriate for standardization.

The SASW technique is based on the measurement of the speed of propagation of surface waves in the material. It allows determination of the thickness of the layer and its stiffness in terms of shear modulus, G_0, under small strains. Its main advantage, in comparison with the other techniques, is the greater depth of investigation that can be achieved and the capability to determine multi-layer stiffness (Nazarian et al., 1987; Roësset, et al., 1989; Stokoe et al., 1999). Moreover, its results can be compared with values of small shear strain modulus obtained in the laboratory. In fact SASW is the only test that will provide continuity among design, laboratory and construction. In addition, giving a practically intrinsic material property, G_0, it can be used to correlate with other test results, mainly with index properties (Gomes Correia et al., 2004a).

It is recommended that the use of these methods should be obligatory to guaranty a uniform stiffness for soils, as well for all the pavement layers and for sub-ballast layers in rail track. Brandl (2001a) reported that with the use of RICCC on high-quality fill materials and intensively compacted embankment settlements may be of the order of 0.1% to 0.2% of its height (H). This is a very important improvement in relation to the local control methods where embankment-settlements obtained are typically of order of 1%.

The rolling dynamic deflectometer (RDD) presented by Lee and Stokoe is a truck-mounted, electro-mechanical system, measuring continuous deflections while moving at approximately 1.6 km/h on pavements (roads and airports). This device has been developed to be used at project level as an alternative to the rolling wheel deflectometer (RWD) moving at 88 km/h that is more adapted for network level. This last device uses laser technology to measure deflections, similar to the high speed deflectograph (HSD) of the Danish Road Directorate, reported by Rasmussen et al. (2002). This same technology could be also be implemented into a moving rail car to measure track modulus.

From this inventory it can be concluded that a lot of in situ tests are being used at the construction and design stages. Unfortunately it must be pointed out that the drawback of most routine in-situ tests lies in the fact that the stress and strain distribution necessary for the identification of constitutive laws is unknown. Consequently only the tests with well-known boundary conditions can be expected to be used universally, while index tests will apply only for the cases for which they were established and different correlations may be required for different materials (Atkinson and Sallfors, 1991).

In practice it is expected that different results can be obtained by these different tests, using different types of loadings (strain rates), different induced stresses and different boundary conditions. This have been reported by several authors (Fleming and Rogers, 1995; Gomes Correia et al, 2004a,b).

Figure 22 form Lee and Stokoe illustrate these realities.

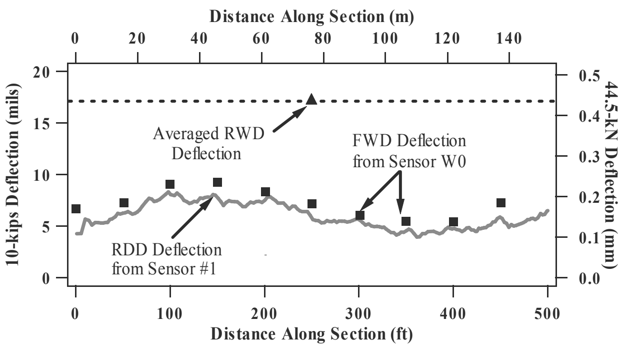


Figure 22. Comparison of results from different field loading tests (Lee and Stokoe and Bay, 2005)

Therefore, any prediction of pavement performance based in direct values obtained by simplified analysis of field test results would be different. A more sound analysis of tests results must be developed, taking into account the non-linear behaviour of soils and unbound materials, as well as strain rate effects (Gomes Correia et al., 2004a,b). Furthermore, the use of field results in design should be corrected to take into account the design moisture content or suction of material for the service life of the structure, as also mentioned by Edil and Sawangsurija.

4 MODELLING AND DESIGN – SERVICEABILTY LIMIT

4.1 *Main framework for modeling and design*

In the process of design, material models are an important component in the response model. Associated with the material models, it is necessary to take into account the tests needed to obtain their parameters. This chain of operations must be always borne in mind at the design stage in order to ensure a good planning of the tests needed for the models. This process will be completed by choosing the performance models and relevant design criteria (Gomes Correia, 2001).

For a more practical application, the complexity of all the process is divided in three levels: routine design, advanced design and research based design. An outline of these levels of design were reported in COST 337 action and summarized by Gomes Correia (2001). The most relevant aspects are: (1) the non-linear material behaviour modelling of unbound granular materials and subgrade soils should be incorporated in layered elastic systems for pavement and rail track response in the routine design level; (2) In heavily trafficked road pavements with structural layers of bituminous materials, viscoelastic models should be used, and viscoelastic multilayer systems could be an option for modelling pavement response; (3) In advanced pavement design and research based design, there is a need to incorporate elastoplastic models, mainly for unbound granular materials (aggregate mixtures and ballasts), in 3D analysis of pavement response. These models must incorporate the several aspects related with permanent deformation: densification, crushing of particles, freeze-thaw cycles; (4) A more fundamental understanding of unbound granular materials should be focused in micromechanics of particulate medium, and for soils in a better knowledge of the effective stresses in non saturated medium; (5) The incorporation of all theses factors, including environmental conditions, must be done using performance models working in an incremental way. In this way alternative materials could be incorporated in design with the same framework of unbound granular materials.

To conclude it must be stressed that verification, calibration and validation are necessary processes for establishing a design method. Verification is intended to determine whether the operational tools correctly represent the conceptual model that has been formulated. This process is carried out at the model development stage. Calibration refers to the mathematical process by which the differences between observed and predicted results are reduced to a minimum. In this process parameters or coefficients are chosen to ensure that the predicted responses are as close as possible to the observed responses. The final process is validation that ensures the accuracy of the design method. This is generally done using historical input data and by comparing the predicted performance of the model to the observed performance. To obtain accurate data of internal stresses and strains, improvements in instrumentation are still necessary.

Based in this general framework, some particular aspects are developed hereafter in relation to the papers dealing with modeling and serviceability design, for pavements and railways.

4.2 *Pavement serviceability design*

Most of the routine structural design methods for flexible pavements are based on an analytical method with design criteria derived from empirical design methods. In the design procedure, the pavement is generally considered as a multi-layered elastic system (response model), generally three layers: the subgrade, the base layer(s), and a top layer which represents all bitumen-bound layers. The layers are characterised by Young's modulus, Poisson's ratio and thickness. The traffic load may be expressed as a standard dual wheel load, and the cumulative number of axles the pavement will have to carry during its service life. Stresses and strains induced in the pavement layers and in the subgrade, and particularly in critical positions (Fig. 23), are calculated by means of commercial computer programs available since the late 1960s. These programs generally represent the pavement structure by a system of homogeneous layers of uniform thickness and infinite horizontal extent, on a semi-infinite, homogeneous subgrade. The layers and subgrade materials are supposed to respond to traffic loading as linear elastic media, which reduce the stress-strain relations to the well known Hooke's law.

The primary design criteria to determine the service life of the pavement are the vertical (compression) subgrade strain and the horizontal (tensile) asphalt strain (performance models). The former is considered to be indicative of the plastic deformations in the subgrade soil and is thus considered as a criterion of serviceability limiting pavement rutting.

The vertical compression strain in subgrade soil is expressed by a relation between the number of strain repetitions (N) caused by the standard design load and the permissible compressive strain in the subgrade. This relationship is given by:

$$Np = \left(\frac{k}{\varepsilon}\right)^{\alpha} \cdot 10^6 \qquad (2)$$

where: Np is the permissible number of load repetitions at a given strain; ε is the vertical elastic strain at the top of subgrade or of UGM (μstrain), and k and α are constants.

Some frequently used relationships are given in Figure 24. It should be noted that these criteria produce very different in-service performances in pavement design (Gomes Correia, 2001). In fact, Shell criterion is mainly related to roughness (Present Serviceability Index PSI=2.5) and the Asphalt Institute (TAI), the Transportation Research Laboratory (TRL) and the Australian State Road Administration (NAASRA) criteria are related with a certain rut depth, with limiting values of 12.5 mm, 10 mm; 25mm, respectively.

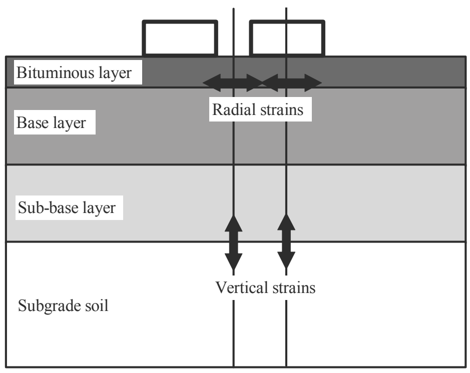


Figure 23. Sketch of a pavement structure and design criteria for two loading types

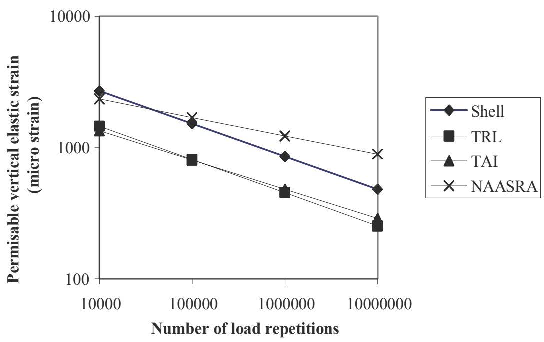


Figure 24. Different subgrade strain criteria

It is interesting to observe, with the relationships of Figure 16, that for 10 million of load cycles the permissible strain level at the top of subgrade will be, depending of the criterion chosen, between 0.04% and 0.1%. According the general framework of mechanical soil behaviour presented by Jardine et al. (1991), these values are surprisingly high for a criterion pretending to avoid the permanent strains. Mainly for the high strain level, the built up of permanent strains and of pore pressure (in fine soils) will be expected, following in the incremental collapse – range C in Figure 17.

It must be noticed that these strain criteria were fundamentally derived for different climatic conditions, different materials and different loading conditions, and cannot, without some form of validation, be transferred to other conditions than those for which they were derived. In consequence, a more fundamental approached is needed to predict directly the permanent deformation in the pavement surface.

Moreover, the experimental evidence that soil subgrade and granular materials in pavements exhibit nonlinear and time dependent behaviour under load, require that these material properties must be considered in pavement modelling to ensure a valid stress, strain and deflection evaluation of these structures.

In fact, deflection of a pavement may be decomposed into an elastic component and a plastic settlement, according the following relationship:

$$\delta = \delta^{e} + \delta^{p} \quad (3)$$

One important feature is that the increment of residual or plastic settlement (i.e. non elastic) over one cycle is much smaller than the amplitude of the elastic deflection, while the amount of such residual settlement accumulated at the end of N load cycles (in pavement can reach several millions) may be the same order as the elastic deflection.

In soil mechanics, various elastoplastic models simulating accurately the cyclic behaviour of soils and granular materials have been developed. For instance, elastoplastic incremental models for soils and unbound granular materials submitted to cyclic loading are developed within dynamic and cyclic engineering framework. However, these models are unable to describe materials submitted to a large number of cycles. Modifications were than proposed to deal with this problem, mainly by kinematics hardening modifications which lead to a better modelling of plastic strains under several millions of cycles (Hicher et al., 1999; Habiballah et al., 2003; Vincens et al., 2003). However, it must be stressed that in the formulation of cyclic constitutive laws for materials, elastic component, mainly for granular materials, must be described by non-linear elastic models representing the observed behaviour (Balay et al., 1997, Gomes Correia et al., 1999; Abdelkrim & Buhan, 2003).

The most advanced codes can also simulate dynamic loadings, crackings, voids beneath surface layers and joints.

There are a number of software packages that can be used for pavement response analyses. The tools available are: (1) 2D (axis-symmetric) Finite Element programs (FEM) and 3D FEM. The main advantage of using a finite element approach is the possibility of using other constitutive models than linear elasticity in both the vertical and horizontal directions, and the possibility of simulating boundary effects and geometry. Important results comparing 2D and 3D analysis can be found in Balay et al. (1997; (2) 3D Finite Difference programs (FDM); these programs are similar to the FEM.

Nowadays these response models are operational to calculate transient displacements, stress and strains under a moving load (Gomes Correia et al., 1999). However, the prediction of plastic deformations by elastoplastic model, or equivalents, are still at research level. The contributions of Abdelkrim et al. and Erlingsson are welcome in this field. The model of Abdelkrim et al., which is already mentioned (Abdelkrim & Buhan, 2003), should have further validation for pavement response under the effects of a moving wheels (see comment in 3.2.4). The model of Erlingsson seems very promising taking into account the good agreement between the calculated stresses and permanent deformations and the measured values in a full scale pavement under simulated traffic by a Heavy Vehicle Simulator (HVS). Figure 25 illustrated the performance of model predictions.

4.3 *Rail track serviceability design*

Brown & Selig (1991) and Lord (1999) summarize the main highlights related to railways foundations. The last author, in his discussion paper to the 12th ECSMFE, emphasized the empirical rules still used at the construction and design levels. He also noticed the importance to consider dynamic aspects in design.

The design criteria of these structures involve environmental aspects related to noise and vibrations, as well as to the settlements of the track.

Rail track design is probably one of the most complex soil-structure interaction problems to analyse. The various elements in design process comprise (Lord, 1999): (1) multi-axle loading varying in magnitude and frequency; (2) deformable rails attached to deformable sleepers with flexible fixings, with sleeper spacings which can be varied; (3) properties and thickness of ballast, sub-ballast, prepared subgrade (if adopted); (4) properties of underlying soil subgrade layers.

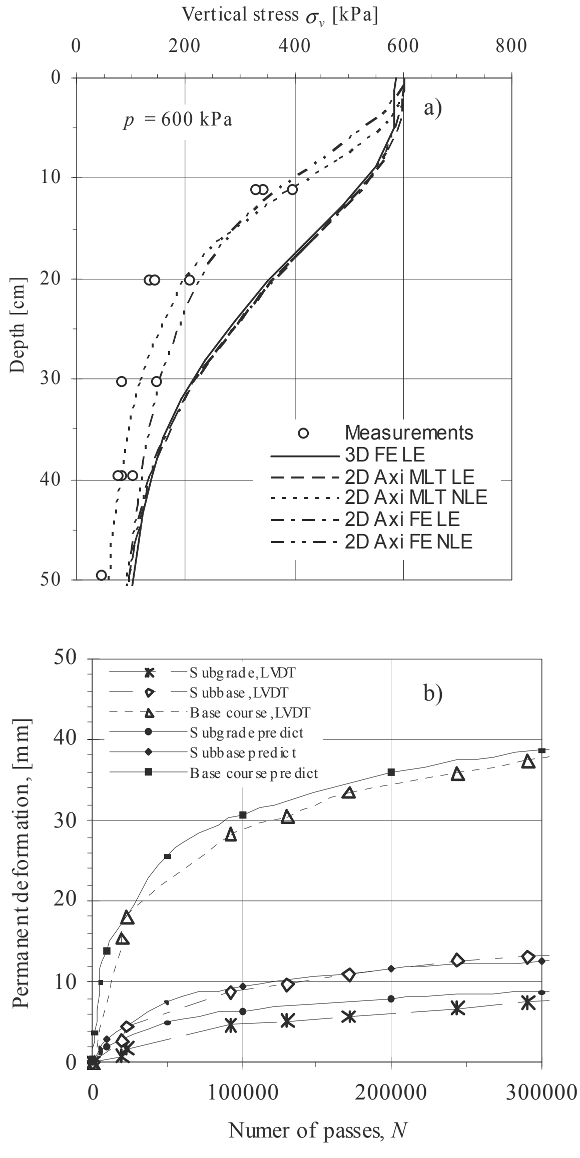


Figure 25. a) Vertical induced stress under the centre of a single tire as a function of depth (axial load: 160 kN; tire pressure: 600 kPa); b) Rutting prediction vs measurements of permanent deformation as a function of load repetition (Erlingsson 2005)

The importance of some of the rail track components related to the track's durability under severe environmental conditions for the service life of the structure should be noted, as mentioned by Nurmikolu and Kolisoja.

As for pavements, at routine design level, several railway track models are operational and some commercially available. The most popular and simplest model for rail track design represents the rail as a beam, with concentrated wheel loads, supported by an elastic foundation. The stiffness of elastic foundation incorporates the sleepers, ballast, sub-ballast and sub-grade, but it is not possible to distinguish between the contribution of the sleeper and underlying layers. This simplified approach has been used to establish dimensionless diagrams in order to quickly assess the maximum track reactions both for a single axle load and a double axle load when the track parameters are changed (Skoglund, 2002).

More sophisticated models have been developed which represent the rails and sleepers as beams resting on a multiple layer system (as in pavements) comprised of the ballast, sub-ballast and subgrade. These models include the commercial programs ILLITRACK, GEOTRACK and KENTRACK cited by Lord (1999).

In these models incorporating multiple layer systems, the design criterion is identical as for pavements, keeping vertical strain or vertical stress at the top of subgrade soil below a determined limit. This criterion is an indirect verification of limited permanent settlements at the top of the system, having the same drawbacks as mentioned for pavements.

Woldringh (2001) summarized some values of allowed permanent deformations adopted in some countries. These are synthesized in Table 4, with other relevant information.

To overcome the drawbacks of the previous models, two direction of advanced modelling are identified. The first category of models aim at improving the theory of beam resting on continuous medium by introducing a spring-dashpot to better simulates a multiple layer system. Furthermore, the model was also improved by introducing a moving load at constant speed and also an axial beam force (Koft and Adam). Figure 26 is a sketch of the model. This model is able to determine dynamic response in different rail track systems due to a load moving with constant speed. A drawback of the model is that it is limited to beams with finite length and consequently only steady state solutions can be provided. The authors applied the model to two experiments and concluded with the need to develop more sophisticate models taking into account nonlinear material behaviours, as was observed to the synthetic pads.

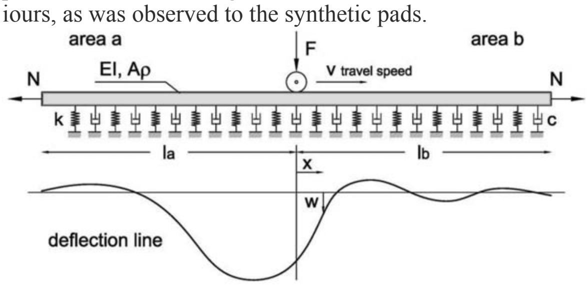


Figure 26. Sketch of a flexible beam resting on continuous spring-dashpot elements loaded by a moving single load (Koft and Adam. 2005).

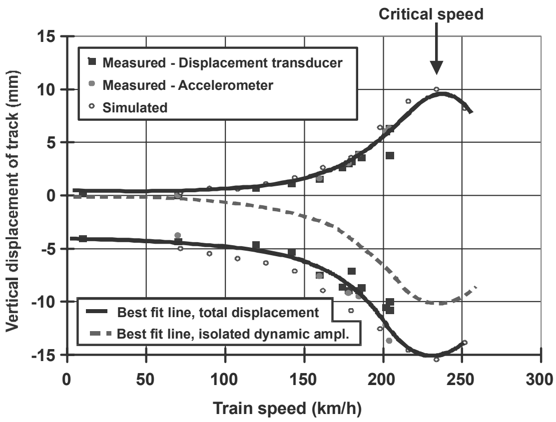


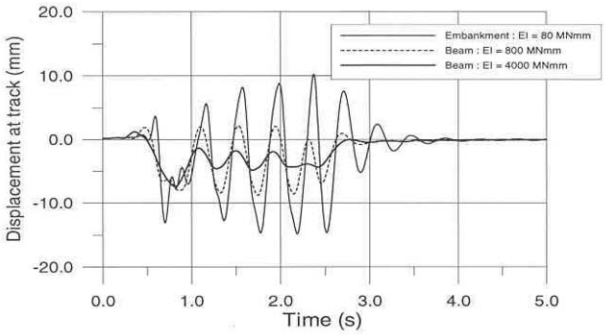
Figure 27. Peak upward and downward displacement of embankment at Ledsgärd, Sweden, vs train speed. Values measured and simulated by VibTrain (Madshus and Kaynia, 2001, cited in Madshus, 2001)

The second group of advanced rail track models use hybrid methods. They couple FEM and multi-layer systems (Aubry et al., 1999; Madshus, 2001). For instance, in rail tracks the track-embankment system is modelled by FEM and the layered ground is modelled through discrete Green's functions (Kaynia et al., 2000). The software developed is called VibTrain. The models referred by Aubry et al. (1999) and Madshus (2001) use

Table 4. Several serviceability criteria for ballast tracks and ballastless tracks

<table>
<tr><th rowspan="2">Country/
Author</th><th colspan="5">Embankment
Residual settlements (after construction) (δ_v (m))</th><th colspan="2" rowspan="2">Rail
Deflection 200kN
($\delta_{v\text{-rail}}$ (mm))</th><th rowspan="2">Others</th></tr>
<tr><th colspan="3">Ballast track</th><th colspan="2">Ballastless</th></tr>
<tr><td rowspan="3">Sweden
(Woldringh,
2001)</td><td>v
100
km/h</td><td>v
160-250
km/h</td><td>v
350
km/h</td><td colspan="2" rowspan="3">-</td><td colspan="2" rowspan="3">-</td><td rowspan="3">-</td></tr>
<tr><td>≤ 0.30</td><td>≤ 0.20</td><td>≤ 0.10</td></tr>
<tr><td>δ_v /l
(l=10m)
≤0.04/10</td><td>δ_v /l
(l=50m)
≤ 0.15/50
to 0.05/50</td><td>δ_v /l
(l=50m)
≤0.02/50</td></tr>
<tr><td>Germany
(Neidhart,
2001;
Woldringh,
2001)</td><td colspan="3">Transition zones
(bridge/embankment)
δ_v /l(=20m)
≤0.03/20</td><td colspan="2">δ_v /l(=10m)
≤ 1/500

≤0.06</td><td>-</td><td>Ballastless
≤1.5</td><td>-</td></tr>
<tr><td>Netherlands
(Woldringh,
2001)</td><td colspan="3">≤ 0.30</td><td colspan="2">≤ 0.03</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>Japan
(Sunaga,
2001)</td><td colspan="3">≤ 0.10</td><td colspan="2">≤ 0.03</td><td colspan="2">≤ 7.0</td><td>-</td></tr>
<tr><td rowspan="2">Austria
(Brandl,
2001b)</td><td colspan="3" rowspan="2">≤ 0.10 to 0.30</td><td>v ≤
160
km/h</td><td>v >
160
km/h</td><td>v ≤
160
km/h</td><td>v >
160
km/h</td><td rowspan="2">Differential
settlements be-
tween rails
≤2mm to 3mm</td></tr>
<tr><td>≤0.06</td><td>≤0.03</td><td>≤1.0 to 2.2</td><td>≤1.5 to 2.0</td></tr>
</table>



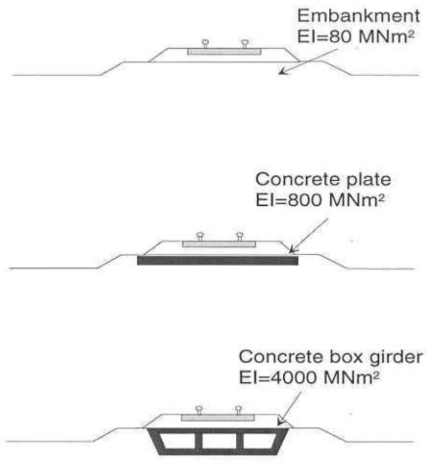


Figure 28. Effect of countermeasure based on longitudinal girder in the embankment, simulated with by VibTrain (Kaynia et al., 2000)

frequency domain having the drawback to deal directly with non linearity behaviour of materials. However, solutions in the time domain also exist (Hall, 2000 – mentioned by Madshus, 2001).

This last family of models are very powerful incorporating track-embankment-ground as they are able to simulate behaviour at all speeds from low up to the critical speed. Figure 27 shows the performance of the model VibTrain in prediction of upward and downward displacements of an embankment versus train speed. Figure 28 illustrates simulation results in different rail track structures corresponding to different conceptual solutions of countermeasure.

As referred by Madshus (2001), these models need further validation by field monitoring. It must also be added that in this context dynamic materials characterization should be strongly encouraged, as well as dynamic field observations, mainly for ballasts.

It must be referred that the framework of soil dynamics is an important basis for railways, but the peculiar aspects involving a very important number of cyclic loads (millions of cycles) is not concerned with the soil dynamic studies done in the field of earthquake engineering. Figure 20 illustrates a particular study for “laboratory” sands. However, in ballast the dynamic aspects are still not well known and this justify one of the ongoing European research program SUPERTRACK (Kaynia, 2003). This program also intends to put into operation a numerical model where, by incorporating the global behaviour of the whole of the railway platform and the supporting soil, will serve to quantify advantages and disadvantages, of the methods used at present in the maintenance of ballast platforms. It will be also possible to quantify and predict the consequences that can have, on this type of structures, the increase of the circulation velocity and the axle load in high speed trains.

This will facilitate, on a medium term, the adoption by the European Union of the adequate strategies to take into account these effects. This more fundamental work will develop tools to capture the main aspects involved with the serviceability of rail tack structures for high speed trains. In this respect allowed permanents deformations area a design criterion. These values are very restrictive and should be carefully fulfilled in order to avoid maintenance actions, critical for the operation of this type of structures.

5 CONCLUSIONS

Knowledge on the behaviour of soils under cyclic loading is an essential part of the foundation design of infrastructure and offshore structures. This includes structures subjected to wave loading, wind, earthquake loading, and traffic and machine vibrations. Structures subjected to wave loading are fixed offshore oil and gas platforms, near shore liquid natural gas (LNG) terminals, anchors for floating structures, harbours, breakwaters, storm surge barriers, etc. Structures subjected to wind loading are wind power structures, bridges and tall buildings. Earthquake loading will influence slope stability, buildings and other structures, both offshore and on land. Traffic and machine vibrations will mainly influence the behaviour of structures and their serviceability. Wave and wind loading will normally have load periods of the order of 10s or more, whereas earthquake, traffic and machine vibration will have lower periods of about 1s or less. The fundamental behaviour of soils is independent of the load period, and the general principles apply independent of period. The numerical values, however, are rate dependant and will be a function of the load period.

(1) Offshore

Cyclic loading generates pore pressure and reduces the effective stresses in soils, causing average and cyclic shear strains that increase with number of cycles. Key issues for the foundation design of offshore structures include bearing capacity, cyclic displacements, dynamic foundation stiffnesses, settlements due to cyclic loading and soil reaction stresses. The following soil parameters are needed to analyze the foundation design aspects of offshore structures: cyclic shear strength, cyclic shear modulus, damping, permanent shear strain due to cyclic loading, pore pressure generation and recompression modulus. These soil parameters can be determined from triaxial and DSS laboratory tests with various combinations of average and cyclic shear stresses. A convenient way to interpret and present the data from cyclic laboratory tests is to plot contour diagrams of average and cyclic shear strains and permanent pore pressure as functions of average and cyclic shear stresses. Such diagrams contain the information required to derive cyclic shear strength and deformation characteristics for foundation design of structures subjected to combined static and cyclic loading. Calculation procedures based on such soil data framework have been verified by back-calculations and predictions of prototype behaviour, and by model tests.

A key aspect of offshore design these recent years has been to find innovative ways to reduce the foundation costs. This trend is expected to continue in the future. Anchors have been used with success in deepwater, as they provide reliable and economical foundation solutions. The reader should read the two keynote papers prepared for the ISFOG conference in Perth, Australia medio September, which present the results of an industry-sponsored study on the design and analyses of suction anchors in soft clays (Andersen et al 2005 on suction anchors, Murff et al 2005 on vertically loaded drag anchors). These two papers, prepared jointly by a large group of engineers and scientists working in the field, represent the state-of-the-art on anchors in clays. The two summarize a number of prediction methods and data related to installation performance and holding capacity of anchors. Research topics with the potential for improving current practice are also identified.

Working at greater and greater water depths has also been the focus over the last years. In less than 20 years, deepwater milestones in the Gulf of Mexico have gone from depths of 300 m to 2000 m. The assessment of "geohazards", or events due to geological features and processes that present severe threats to humans, property and the natural and built environment, has rapidly become a requirement for foundation design in deepwater. The triggers for underwater slope stability can be natural on-going geological processes or human activities. Triggers include earthquake activity, rapid deposition, excess pore pressures, toe erosion or top deposition, sensitive and collapsible soils, melting of gas hydrate, active fluid/gas flow and expulsion, mud volcano eruptions and diapirism and sea level lowering during glacial periods. Predicting the hazard posed by geological processes, and evaluating the human, environmental and economical consequences of geohazards require that each of the uncertainties in the situation to analyse are quantified and addressed properly. The integrated evaluation by geologists, geophysicists and geotechnical engineer is essential. A state of the art study was made for the extremely large underwater Storegga slide where the offshore Ormen Lange gas field is located. The results have been compiled by Solheim et al. (2005) in the thematic volume of Marine and Petroleum Geology. The positive conclusion of this intensive work made it possible to approve the development of the field.

(2) Transportation

An urgent mutation of conventional characterization and specifications of subgrade soils and unbound granular materials for pavements and railways is necessary to pursue the changes in advance design and new construction technologies. Moreover, it is also necessary to optimize the use of traditional materials and allow the utilisation of new materials. In this context the use of performance based tests should be more used in practical applications. At laboratory level the precision cyclic triaxial test seems to be a good compromise, facilitated by the existing new CEN standard. The test protocol allows to determine parameters for modeling as well for ranking materials based in the resistance to permanent deformation. Physical model tests are also very useful mainly those simulating traffic loading by a moving wheel; these tests are more representative of loading conditions inducing the rotation of planes of principal stresses, not possible by the cyclic triaxial test. Permanent deformations under traffic loading will be more representative by this type of physical tests and consequently more appropriated for calibration of numerical models and validation of design methods.

At field level and particularly at construction quality control it is encouraged to move from local index tests (spots) to mechanical continuous measurements, such as roller-integrated continuous compaction, “portancemeter” and spectral analysis surface wave (SASW). This last technique is very promising since is the only field tool providing continuity between the design, laboratory and construction. To evaluate the bearing capacity of pavements and railtracks after construction rolling dynamic deflectomters using laser technology should become routine tests.

There exist a lot of in situ tests to evaluate stiffness of materials using different induced stress and strain levels, as well as load and strain rates. The analyses of these tests require sound theories incorporating non-linear behaviour of soils and granular materials, as well the state conditions of the materials. The application of these results to design need corrections to taken into account the representative environmental conditions prevailing during the service life of the structure.

There is an urgent need to move from the empirical rules used in routine design of pavements and rail tracks to a mechanistic approach making use of the knowledge of the mechanical behaviour of soil mechanics framework, non saturated soil mechanics and soil dynamics.

Open questions and further developments:

Research needs of anchors include several aspects (Andersen et al. 2005), and only key questions are addressed herein.

During the installation of suction anchors, the profession should establish an agreed basis for the safety factor against plug failure during installation. Acceptable design values for the safety factor should also be determined. As the penetration resistance and the required penetration underpressure depend strongly on the remoulded shear strength, research should be done to establish the optimum method of evaluate this parameter. Bearing capacity factors should also be determined for different geometries and boundary conditions of internal stiffeners and external protuberances. Moreover, studies should be made on the interface shear strength of carbonate rich soils, both in terms of the radial stress and the interface friction angle.

With respect to the bearing capacity of suction anchors, further work should address the uncertainties related to whether a crack will develop or not along the active (windward) side of a suction anchor. Set-up along the skirt walls should be documented with relevant experimental data, both from model tests and prototype data. The effect of large displacements at failure should also be investigated, as well as the effect of strain softening on the anchor capacity. Since there is today no consensus on the safety factors to be used for suction anchors, this aspect should be carefully considered.

For vertically loaded drag anchors, validation of the results obtained analytically should be done with comparisons of analytical results with model and prototype observations.

The main challenges today in the field of offshore geohazards are related to risk assessment for deep and ultra-deep sites. In addition, although the knowledge, technology and tools required for deep and ultra-deep water site investigations have improved significantly over the past decade, there still remains a need for improvements to reduce the uncertainties in the geohazards assessment (e.g. Nadim and Locat, 2005):

- Geophysical, geological and geotechnical site investigation techniques
- Effects of sample disturbance
- Location and quantification of gas hydrates, and their effects on soil behaviour
- Presence of gas and their effects on soil behaviour
- Tools and methodology for prediction and measurement of pore pressures
- Interpretation of sediment rate, stress conditions, pore pressure conditions
- Field measurements and monitoring of geohazards
- Methods and techniques for early warning
- Analysis tools for assessment of seabed instability and failure probability
- Quantification of uncertainties in analysis parameters
- Material models, mechanical models and analysis tools for stability assessment
- Slope stability and dynamic slide mechanisms, including progressive and retrogressive failure
- Effect of earthquake loading on soil strength and post-earthquake stability
- Analysis tools and methods for evaluation of consequences, e.g. slide dynamics and run-out, slide velocity and impact, and tsunami generation, impact and run-up

In analyzing geophysical .tests, such as bender elements in laboratory or SASW in the field where is the limit of applying mechanics of continuum mechanics to particulate medium?

In soil dynamics the damping ration has been studied for many soils. However, the behaviour of very dense unbound granular materials is less known. Moreover, the effect of a big number of cycles (millions) should be considered, particularly their influence in the damping ratio.

A more fundamental approach for granular materials should be focused in micromechanics of particulate medium, incorporating the glue created by capillarity stresses in non saturation conditions of the material.

There are a lot of different mechanical field tests to evaluate construction quality control. What is the best test relating field, laboratory and design?

The serviceability criteria in routine design of pavements and railtracks are of empirical basis. The use of new materials, different loading conditions, different environmental conditions and new technologies in construction require design methods based in a mechanistic-performance basis. For pavements mechanistic models should be developed considering the peculiarities of soils and unbound granular materials: non linearity, pre-straining, state conditions, effective stress concept; number of cycles. For railtracks dynamic aspects should be considered (integrating ground, embankment and track), as well as residual settlements very restrictive for high speed trains.

Material models and structural models should be calibrated and validated for design by full scale tests with specific instrumentation. Vibration measurements in ballast are of relevance to understand its performance and conclude about countermeasures applications.

In which concerns railtracks for high speed trains a question remain: what are the conditions dictating the choice of a ballast track or a ballastless track?

LIST OF PAPERS IN SESSION TS2E

Abdelkrim, M., Bonnet, G., de Buhan, P.: Modelling the residual settlement of geotechnical structures submitted to long term cyclic loading.

Badv, K.: Some geochemical and mineralogical characteristics of Urmia Lake deposits.

Calle, E., Sellemeijer, H., Visschedijk, M.: Reliability of settlement prediction based on monitoring.

Cheuk, C.Y., White, D.J., Bolton, M.D. Deformation mechanisms during uplift of buried pipes in sand.

Edil, T.B., Sawangsuriya, A.: Earthwork quality control using soil stiffness

Elmi, F., Favre, J.-L.: Penetrometer interpolation using seismic data for offshore application.

Erlingsson, S.: Numerical modelling of unbound granualar materials in thin pavements structures.

Gnanendran, C.T.: Effects of combined cyclic vertical and horizontal loading on unsealed airstrips: model study.

Gomes Correia, A., Anh Dan, L.Q., Koseki, J., Tatsuoka, F.: Stress-strain behaviour of compacted geomaterials for pavements

Hamre, L., Bye, A., Søreide, O.K., Athanasiu, C.: Study of transient pore pressures due to cyclic loads to optimize the foundation concept for Sakhalin Platforms.

Hayano, K., Kitazume, M.: Stress distributions and its evaluation in asphalt pavement ground subjected to roller loading.

Kikuchi, Y., Otani, J., Mukunoki, T., Yoshino, H., Nagatome, T.: Permeability on lightweight treated soil mixed with air foam.

Kliner, J.T.R., Grozic, J.L.H. A new method to estimating gas hydrate content in soil specimens.

Kodikara, J., Ranjith, P.G.: Modelling of salt migration in stabilised pavement materials.

Kopf, F., Adam, D.: Dynamic effects due to moving loads on tracks for highspeed railways and on tracks for metro lines.

Laue, J., Springman, S.M., Nater, P., Graemiger, E., Ducksch, A. Creating seafloor conditions in geotechnical centrifuges

Lee, J.L., Stokoe, II, K.H., Bay, J.A.: The rolling dynamic deflectometer: A tool for continuous deflection profiling of pavements.

Micic, S., Lo, K.Y.: Increasing pullout resistance of offshore foundations in soft clays

Momoya, Y., Sekine, E.: Deformation characteristics of railway asphalt roadbed under a moving wheel load.

Nurmikolu, A., Kolisoja, P.: Extruded polystyrene (XPS) foam frost insulation boards in railway structures.

Puech, A., Dendani, H., Meunier, J., Nauroy, J.-F.: Geotechnical characterisation of Gulf of Guinea deepwater Sediments.

Queiroz, R.C., Macari, E.J.: Method for estimating railroad track settlements due to dynamic traffic loads.

Sargand, S.M., Kim, S.-S.: Forensic study of the Ohio SHRP Test Road U.S. 23 flexible test pavement.

Singh, B., Datta, M., Gulhati, S.K. Suction development during pullout of superpile anchors in soft saturated clay.

Strout, J.M., Sparrevik, P.M.: Multilevel subsea piezometer system.

Tufenkjian, M.R., Thompson, D.: Shallow penetration resistance of a minicone in sand.

Vallejo, L.E., Chik, Z.: The evolution of crushing in granular materials and its effect on their mechanical properties.

Valore, C., Ziccarelli, M.: Safety appraisal and rehabilitation of a quay wall.

Vanden Berghe, J-F., Cathie, D., Ballard, J-C.: Pipeline uplift mechanisms using finite element analysis.

Vinogradov, V.V., Yakovleva, T.G., Frolovsky, Yu.K., Zaitsev, A.Al.: Evaluation of slope stability of railway embankments.

Wijeyakulasuriya, V., Ramanujam, J., Creagh, M., Soward, G.: Performance of some unbound roadbase materials form Queensland.

Wilkinson, A., Haque, A., Kodikara, J., Adamson, J., Christie, D.: Physico-chemical aspects of cementitious slurry design for the stabilisation of reactive soils.

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