

Observational Method applied to the Rio Grande Port Breakwater

F. Schnaid, L.G. Mello, S.S. Sandroni

Abstract. A case study describing the experience in modeling and designing the process of construction of the Rio Grande Breakwater on soft clay deposits is summarized in this paper. Field performance during and after embankment construction was monitored with inclinometers, magnetometers and electrical piezometers providing the necessary information to check the design hypothesis, to evaluate the uncertainties related to the natural ground variability and to ensure that the work conformed with acceptable limits of behaviour. The importance of modeling the construction by finite element analysis as an interactive process supported by observations collected from the construction phases is highlighted. The successful completion of the work stimulates the use of the Observational Method in geotechnical practice and, for this reason, guidance is provided for future work.

Keywords: observational method, soft clay, instrumentation, numerical analysis.

1. Observational Method: Design Principles

As reported by Peck (1969) “observational methods have always been used by engineers working in the fields now included in applied soil mechanics, but the observational method – OM is a term having a specific restricted meaning”. The systematization of the OM is associated by Peck to Terzaghi (1961), who reports that between 1912 and 1922 the Swedish State Railroads was using the observational procedure on a large scale in earthworks. Terzaghi (1961) presents some of his works conceived to follow this procedure. In every case reported by Terzaghi the OM was used to compensate for the uncertainties associated to the interpretation of subsoil exploration.

Peck (1969) quotes an early version of the introduction to the book *Soil Mechanics in Engineering Practice*, in which Terzaghi (1948) wrote “the results of computations are not more than working hypothesis, subject to confirmation or modification during construction. Soil mechanics as we understand it today, provides a method which would be called the experimental method”. Reference is made to the practical application of the learn-as-you-go method, closing the gap in knowledge and, if necessary, leading to modifications of the design during construction.

There is little doubt that Peck (1969) framed the ideas and concepts being used, inclusive by Terzaghi, proposing that “the complete application on the method embodies the following ingredients:

- Exploration sufficient to establish at least the general nature, pattern and properties of the deposits, but not necessarily in detail;
- Assessment of the most probable conditions and the most unfavourable conceivable deviations from these

conditions. In this assessment geology often plays a major role;

- Establishment of the design based on a working hypothesis of the behaviour anticipated under the most probable conditions;
- Selection of quantities to be observed as construction proceeds and calculation of their anticipated values on the basis of the working hypothesis;
- Calculation of values of the same quantities under the most unfavourable conditions compatible with the available data concerning the subsurface conditions;
- Selection in advance of a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis;
- Measurement of quantities to be observed and evaluation of factual conditions;
- Modification of design to suit actual conditions”.

In the conclusion of his Rankine Lecture, Peck (1969) mentions that the successful use of the OM is associated to the possibility of optimizing/altering the design during construction, as well as the necessity of having contracts that allow for these changes, without burdening either party involved in an unbalanced way.

Lambe (1973) in his Rankine Lecture focuses on discussing predictions in geotechnical engineering. In his proposal, a classification of predictions as related to when and based on what data was available at this time, named Type B prediction as “made during the construction and would have available data obtained during the initial parts of the construction, such as measurements made during excava-

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tion, foundation construction, etc.”, with the outcome of the event being predicted still unknown.

There is a clear link of Lambe’s concerns of accuracy of predictions with the concepts of the OM, as “the evaluation of a prediction consists of an examination and interpretation of the prediction in the light of the known outcome of the predicted event, it is built round a comparison of the predicted performance with the measured performance”. Specific mention is done to the fact that an engineer “makes decisions and takes actions on the basis of his re-examinations. The Terzaghi-Peck OM depends on an evaluation of predictions” is presented.

Numerous authors have worked within these concepts and developed parallel thoughts from then on, and the OM has also been recognised as a design method in codes like Eurocode 7 (1997). Design review during construction is specifically mentioned.

Requirements to be met are postulated in this Code as reproduced below:

- Acceptable limits of behaviours are established;
- The range of possible behaviour is assessed and is documented that there is an acceptable probability that the actual performance will be within the acceptable limits;
- A monitoring program is devised to document the actual performance. The monitoring program shall document the accepted performance early in the construction process, with data acquisition at sufficiently short intervals as to allow for prompt contingency actions to be undertaken successfully;
- The response time of the instruments and the procedures for analysing the results shall be sufficiently rapid in relation to the possible evolution of the system;
- A plan of contingency actions shall be prepared, to be adopted if the monitoring reveals behaviour outside acceptable limits.

In 1999, Nicholson *et al.* published a relevant contribution to the theme, updating it to modern society’s concerns and requirements, mainly aiming at responding to cost savings, increase in safety and team co-operation as embodied in modern contract types. Their definition is: “the OM is a process in which acceptable limits of structural and geotechnical behaviour are established. In addition, performance predictions, monitoring, review and modification plans, and emergency plans, are fully prepared. The design is checked for robustness before construction starts. During (and after) construction, the results from the monitoring are reviewed against the predictions and robust modifications are introduced where appropriate”.

To illustrate these points, a flow chart of the OM is presented and reproduced herewith (Fig. 1). Risk management, in its technical aspects, is intrinsically linked with and part of the OM. The definition of the OM that encompasses these ideas is properly summarized as: “a continuous, managed, integrated, process of design, construction con-

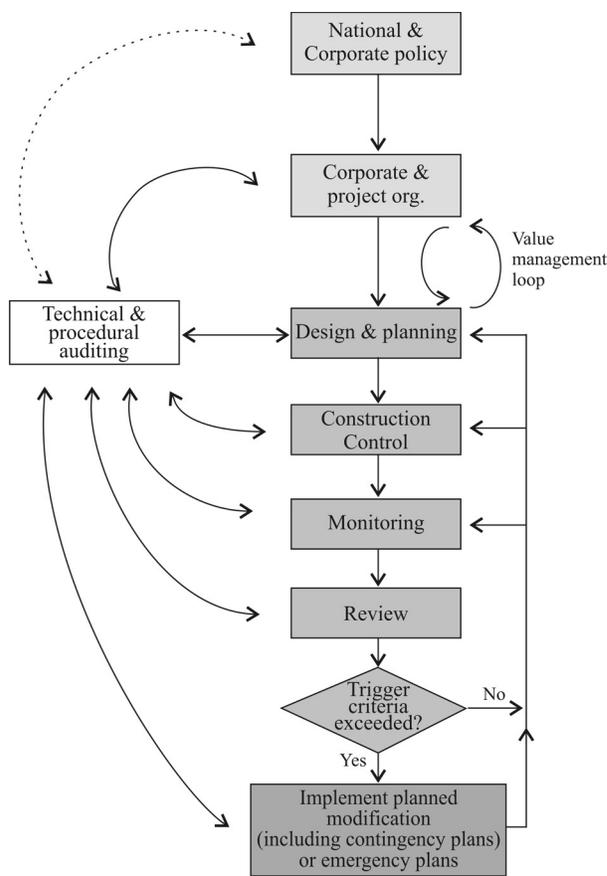


Figure 1 - Observational Method Flow Chart, apud Nicholson *et al.*, 1999.

trol, monitoring and review that enables previously defined modifications to be incorporated during or after construction, as appropriate” (Nicholson *et al.*, 1999). The authors would add the need of previously also approve the modifications to eventually incorporate in design.

As monitoring is an important item of the OM, the postulation of trigger criteria is also implicitly important, to start implementation of planned modifications and/or of the contingency plan if an emergency situation is foreseen.

The postulation of values of forces/stresses and/or movements/displacements, leading to the calculation of velocity of changes and many other techniques of assessing and interpreting the behaviour of a structure, require that all the correct possible models of collapse that a structure in a certain subsoil stratigraphy may undergo are properly identified, that all the geomechanical properties of the distinct material layers are properly determined, and that design calculations, from which the trigger values are derived, are developed with models that correctly simulate the field reality in time/stress/time path.

Numerous Symposiums, Specialty Sessions, Work Groups etc were organized to discuss and present case histories where the OM was used, leading to the existence of quite a number of published technical papers on the topic.

The identification of existing risks in any project, their mitigation during design, and their monitoring throughout construction, has become part of most important works.

Patel *et al.* (2007) refer to a publication by the GeoTechNet (2002) which showed that the implementation of the OM in Eurocode EC7 had shortcomings, as there was a general lack of understanding of the principles of use of the OM, its use within contractual framework on an engineering project and the important responsibilities incumbent on all parties involved, the client, the designer and the contracting teams when implementing the OM approach to a project.

The important discussion that, while in traditional ground engineering projects, monitoring plays a passive role to check original predictions and provide confidence to third party checkers, in the OM monitoring plays a very much proactive role in both design and constructing, allowing pre-planned modifications to be carried out within an agreed contractual framework.

The approach to the application of the OM has developed and matured since Peck (1969), as discussed by Nicholson *et al.* (1999). Peck had adopted the most probable design, evolving to reduced moderately conservative design parameters if and when triggers in the monitoring program were exceeded. Nicholson *et al.* propose a safer approach to design adopting a progressive modification of the design starting based on moderately conservative parameters and then, backed by monitoring, developing to most probable conditions. In this modern approach, risk analysis and risk management became a must, especially with lump sum contracting. Managing geotechnical risks is the focus of many professionals and applied in most important heavy infrastructure projects (*e.g.* Clayton, 2001).

In parallel, the collapse of the Heathrow tunnel and the consequential investigation by the British authorities triggered the incorporation of a Code of Practice (2006) agreed with the risk takers, the Insurance industry, to identify and mitigate risks during design and construction of tunnels, an important area of civil engineering works which uses the OM method approach applied to tunnelling practice, mainly in the New Austrian Tunnelling Method – NATM.

The modification of moderately conservative predefined design to the most probable situation reduces the uncertainties and, therefore, greater site controls are necessary, balanced by rigorous monitoring and existence of proper contingency plans fully discussed previously to works commencement.

Flexibility is required at the work fronts in order to accommodate changes in design and programme. The stakeholders need to be fully tuned with the technical and commercial risks linked to any contingency materializing.

Monitoring becomes a crucial item in the whole construction planning and development. A competent regime

has to be set in place, checking, reviewing and responding to any result in a short time.

The possibility of having brittle behaviour in the structure or rapid uncontrollable deterioration in the materials which does not allow sufficient warning to implement planned modifications have to be excluded by design decisions, as stated by de Mello (1977).

Finally, the authors consider valid to comment the recent use of a modern terminology for the OM: the Interactive Design. In our understanding it is the OM as discussed by Peck (1969) adapted to the third millennium societies' terminologies and needs. Many other designations have been used since Peck presented his Rankine lecture, like Experimental Method and Design-as-you-Go (Ladd, 1991; Staverson, 2006; Negro *et al.*, 2008).

The present paper aims at sharing knowledge of the current OM state-of-practice based on a critical overview of the Rio Grande Port breakwater construction case study. Fundamental concepts that frame the OM - introduced above - have been fully implemented given room to highlight advantages, limitations and perspectives of this type of approach in geotechnical practice regarding ultimate limit states.

2. Rio Grande Port Breakwater: Case Study

The case study describes the stage construction of an extension of a 20 m high marine breakwater laying over a thick soft sedimentary deposit at the Rio Grande Port in southern Brazil. The Western and Eastern breakwaters are 700 m and 370 m long extensions of existing facilities constructed in 1910 by *Compagnie Française du Port Rio Grande* (Fig. 2). The construction completed in March 2011 deepened the existing navigation channel from -14 m to -18 m in order to allow access for larger ships.

Breakwater design was an engineering challenge due to the combination of difficult geotechnical and environmental conditions in the region, with strong currents from the lagoon system, severe winds and large waves. The crest level is relatively low, namely TAW + 5 m, leading to large overtopping during storms. Designed to withstand the im-

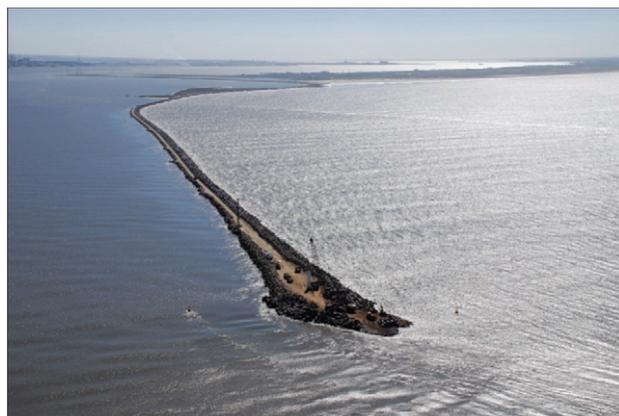


Figure 2 - Eastern breakwater at the Rio Grande Port, Brazil.

part of overtopping waves, the breakwater was constructed in a 1V:1.5H embankment slope, protected by an armour layer and underwater equilibrium berms for slope stability (Fig. 3).

Construction of the extension of the breakwaters was completed in 4 stages: (i) placing a first mattresses layer up to TAW -11.0 m, (b) construction by ships and barges up to TAW -5.0 m, (c) land construction up to TAW +3.0 m and (d) finishing at TAW + 5.0 m during placement of armor layer and tetrapods.

3. Site Characterization and Instrumentation

A preliminary site investigation was carried out at an early design stage and included SPT's boreholes, vane tests and undisturbed soil samples from 4" Shelby samplers. A complementary, comprehensive offshore site investigation campaign was performed from a submersible unit. A series of piezocone tests were carried out along the eastern and western breakwater plan area and undisturbed sampling were retrieved for laboratory triaxial and oedometer tests

(e.g. Schnaid, 2009). Characteristic features of a continuous profile from CPTU data are shown in Fig. 4, revealing a sedimentary deposit with a 3 m sandy-clay layer overlain a 12 m thick soft clay layer. A superficial thin silty-clay layer, recently deposited, is frequently observed along the site. A representative profile of the Eastern breakwater is shown in Fig. 5. Design parameters assessed from laboratory and in situ testing are summarized in Table 1.

Given the challenges arisen by adverse geotechnical conditions, the breakwater construction was instrumented and the OM approach fully implemented. Seven opened 0.8 m diameter steel casing instrument towers were deployed on the seabed, adjacent to the projected breakwater toe contour and embedded into the equilibrium berms. Four towers were located along the Western side (MO01, MO02, MO03 e MO04) and three along the Eastern side (ML01, ML02 e ML03) as shown in the aerial photograph and plan view in Fig. 6. These instrumented towers enabled the instrumentation to be installed and provided protection to the instruments during construction. Instrumentation

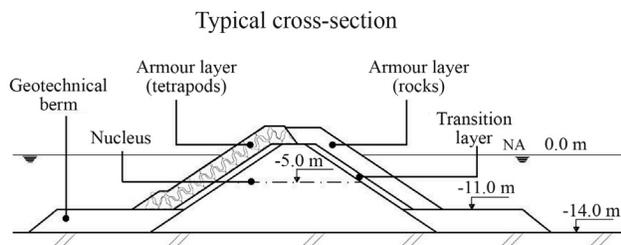


Figure 3 - Layout of a typical cross-section of the Rio Grande breakwater.

Table 1 - Material properties.

Material	Thickness (m)	e_o	C_c	ϕ'
Rockfill	22	incompressible		45°
Very soft silty-clay	4	3.6	1.6	33°
Loose sand	6	2.0	1.0	45°
Soft clay	11	1.0	0.5	26°
Very dense sand	∞	incompressible		45°

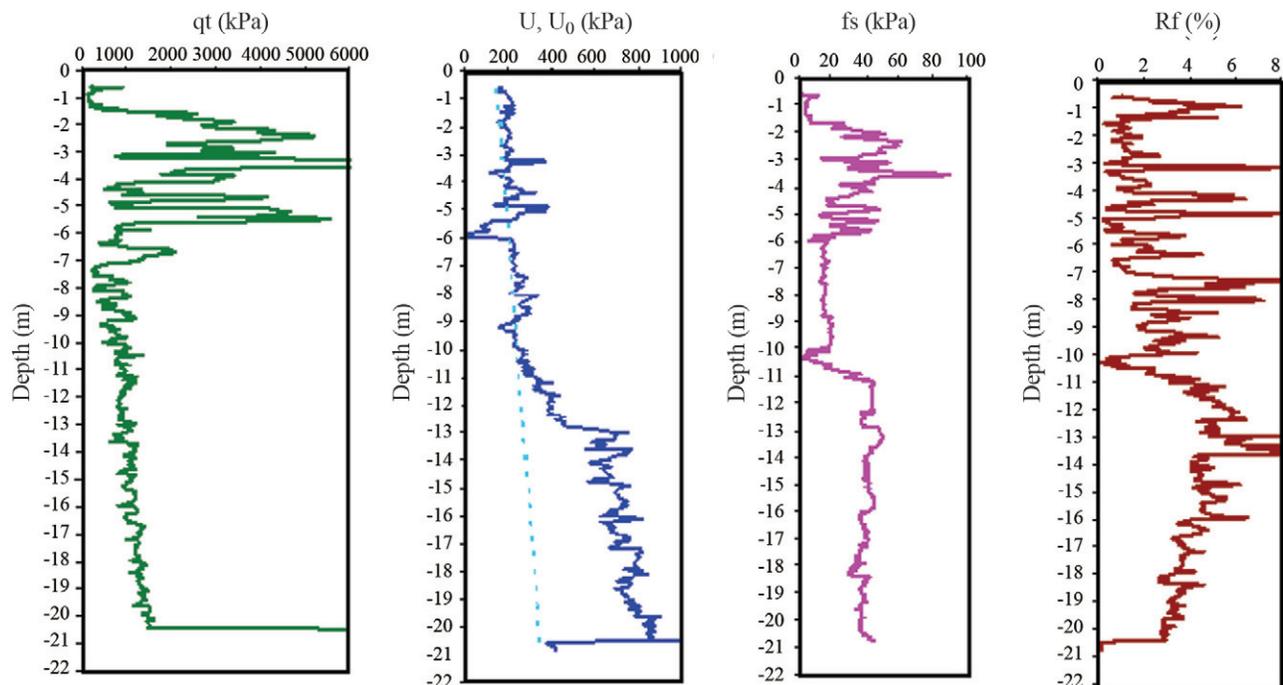


Figure 4 - CPTU testing data at the Eastern Breakwater.

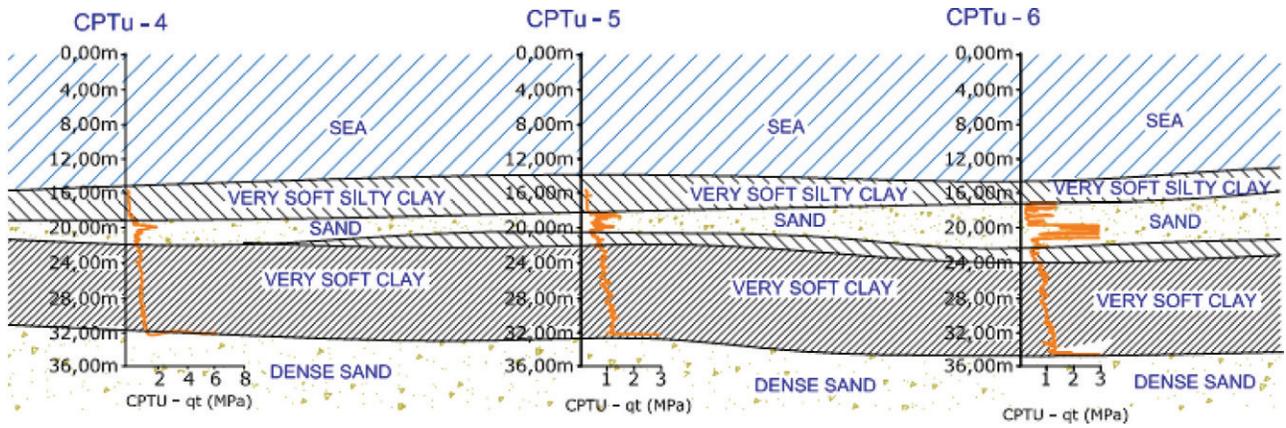


Figure 5 - Soil profile of the Eastern Breakwater.

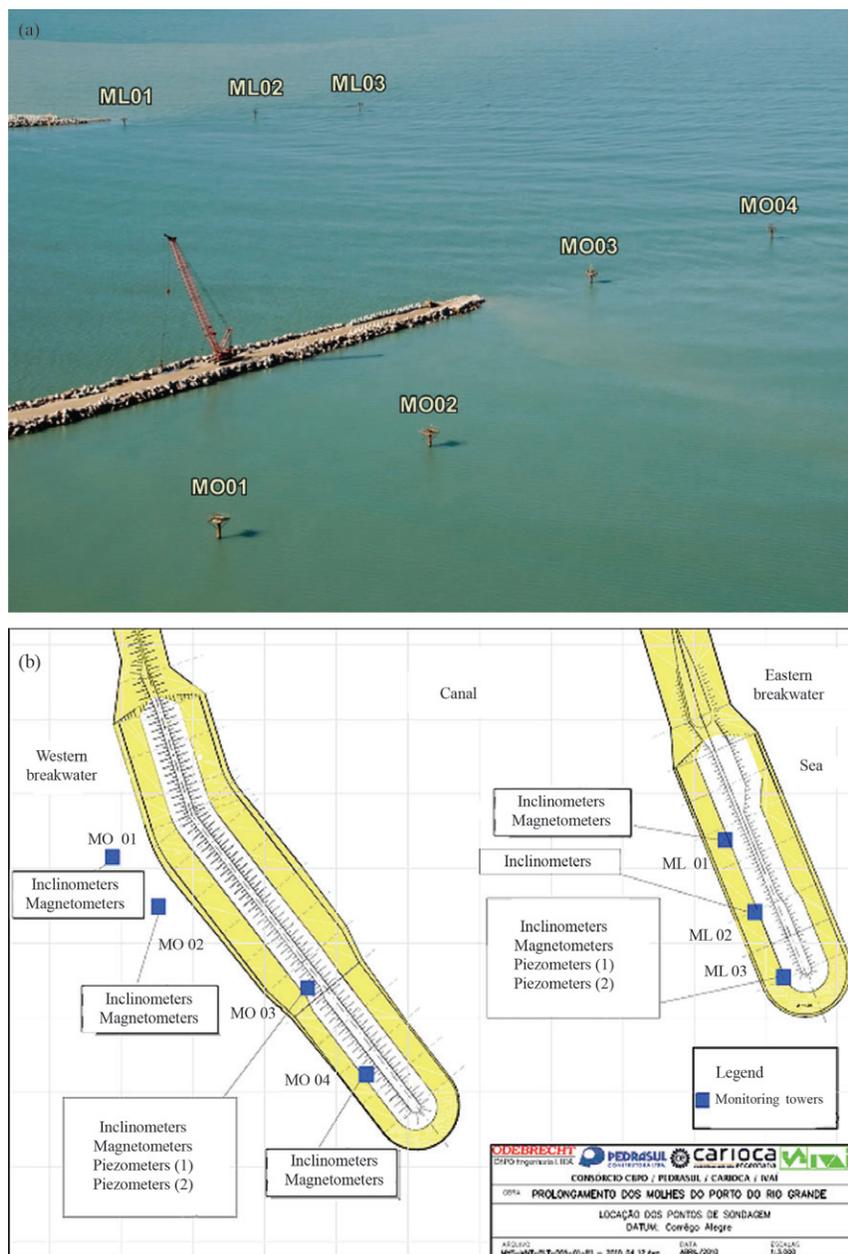


Figure 6 - Location of instrumented sections (a) aerial photograph and (b) plan view.

comprises inclinometers, settlement detection devices (magnetometers) and electrical piezometers. A detailed description of instruments and installation procedures has been reported by Rabassa (2010).

In general, all instrumented towers produced the same qualitative information and, therefore, results from a single instrumented location (MO03) are used to evaluate the measured performance of the breakwater construction. Representative of the overall measured behavior and devised as part of a detailed monitoring scheme, these results are analyzed on the basis of acceptable limits and contingency action plans. For example, horizontal displacement versus depth curves measured at an axis perpendicular to the breakwater for a number of load increments are shown in Fig. 7. Maximum horizontal displacement measured at a depth around 24 m to 28 m reached 140 mm.

The results as presented in Figs. 8 and 9 show the vertical deviation θ and variation of vertical deviation with time ($V_\theta = \Delta\theta/\Delta t$) for a depth of 27 m (depth of large observed displacements). The vertical deviation is defined as the increment in horizontal displacement $\Delta\delta_h$ divided by the distance between the measured points Δz , that is $\theta_v = \Delta\delta_h/\Delta z$. The evolution of θ_v and V_θ with time and cumulative load (ton) reveals aspects of behavior that deserve close consideration. By increasing the elevation of the breakwater in the vicinity of the inclinometer location, both

θ_v and V_θ increase considerably as comprehensively reported in the literature for embankments constructed close to undrained conditions (e.g Ladd 1991; Almeida 1996; Brugger et al., 1999; Almeida et al., 2010). The onset of increasing displacements in August 2009 shown in the figure gives a threshold point in a plane of maximum shear strains at the depth associated to the potential failure surface (see Fig. 9). However, at a constant load, the consolidation process starts and produces a further increase in θ_v and a continuous reduction in V_θ . Clearly vertical deviation reflects the displacement path produced by both undrained shear and consolidation and for this reason it cannot be used alone to define reference acceptable limits of performance in cases where load increments are superimposed to some consolidation (as often observed in practice).

Results from the variation of pore-pressure measurements with time are illustrated in Fig. 10. Recorded measurements show fluctuations of the order of 5 kPa, corresponding to tide and wave oscillations of about 1 m of water column. Within the clay layer, pore water pressures increased significantly during August and September 2009, a period that corresponds to breakwater elevation from -11 m to -5 m below average sea level. In the remaining time, there are periods of pore pressure increments (construction stages from -5 m to +5 m) followed by pressure decrease due to consolidation.

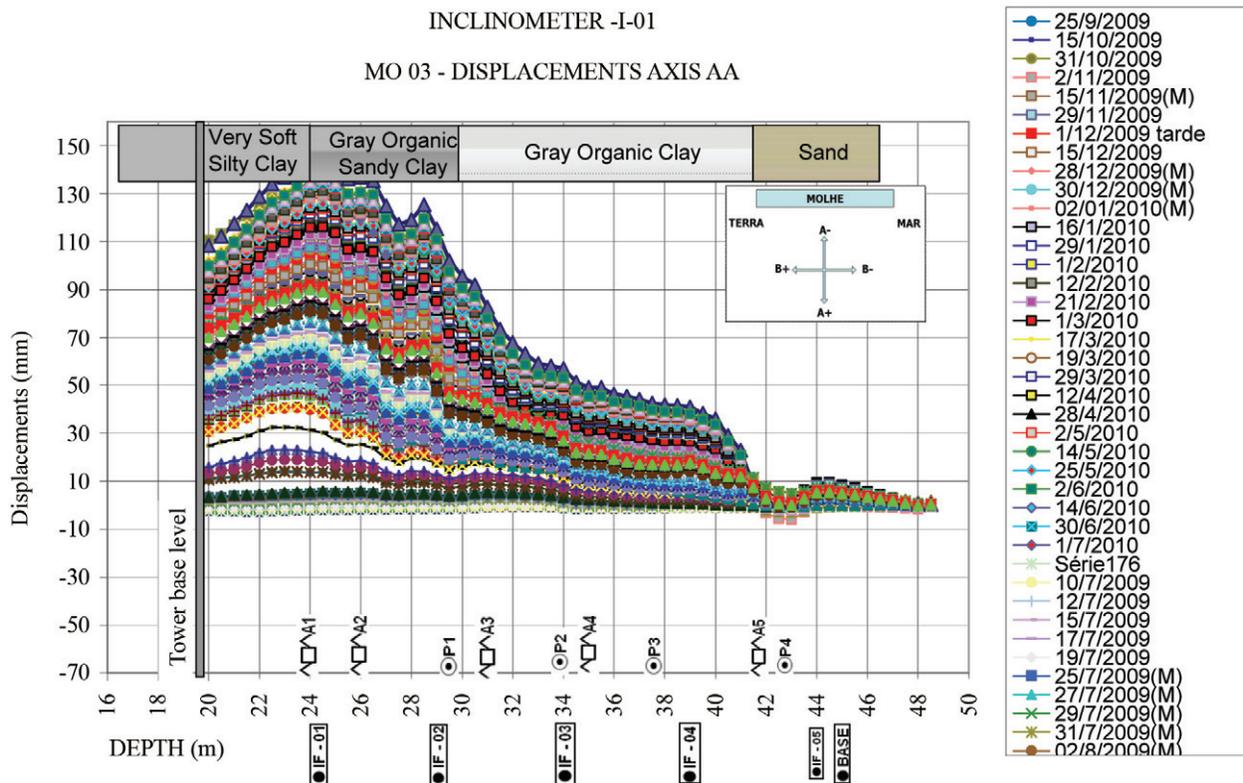


Figure 7 - Inclinometer data for toe embankment position MO03.

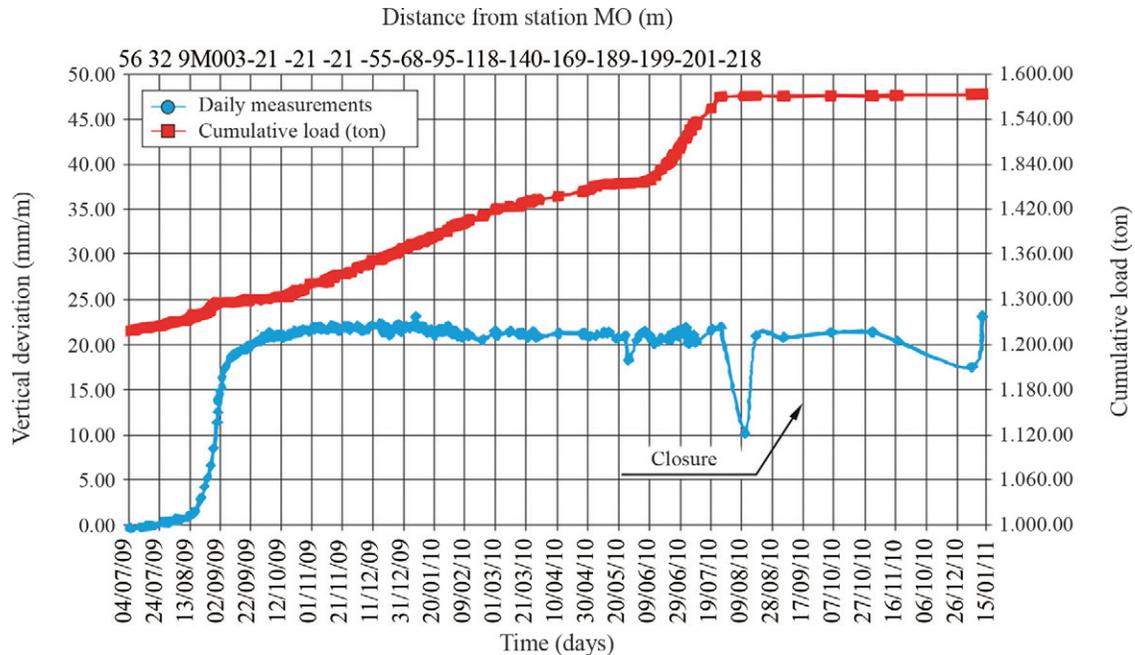


Figure 8 - Vertical deviation and embankment cumulative load plotted against time at a depth of 27 m.

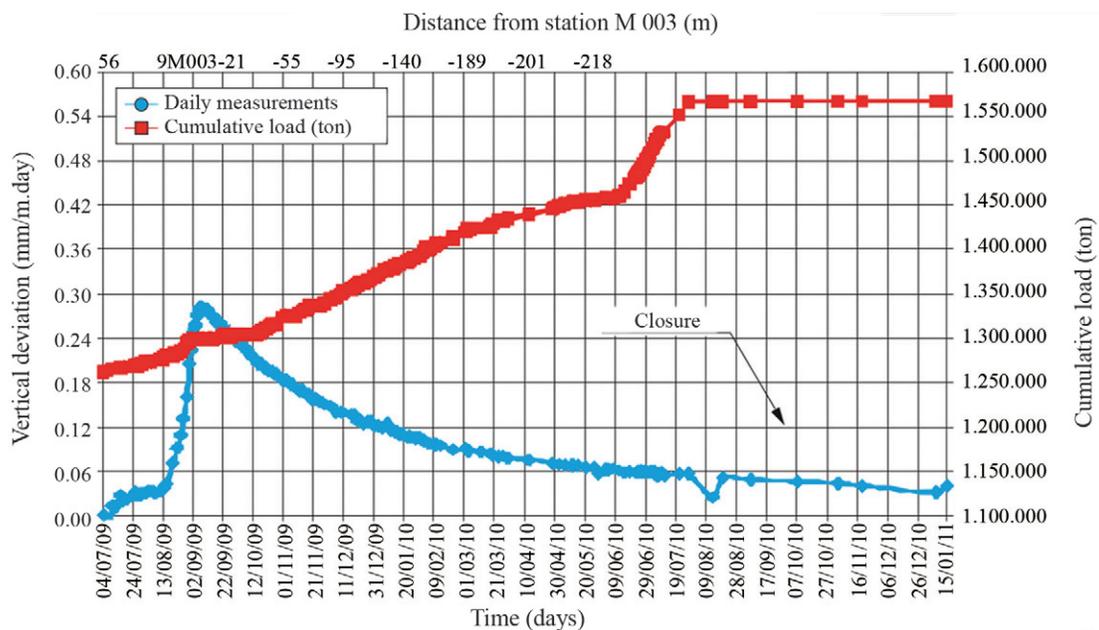


Figure 9 - Variation of vertical deviation and embankment cumulative load at a depth of 27 m.

Acceptable limits of behavior for stage construction close to undrained conditions were defined from results of numerical analyses of representative cross-sections (later summarized by Dienstmann, 2011). Limits conceived to increase rate of monitoring and prepared to implement contingency were defined from both experience (Almeida 1996; Brugger *et al.*, 1999; Almeida *et al.*, 2010) and numerical (Dienstmann, 2011) analyses: (a) vertical deviation θ greater than 15 mm/m.day and (b) variation of vertical de-

viation greater than 20 mm/m. Contingency actions planned to be triggered when monitoring values were outside acceptable limits comprised reducing construction rate, stopping construction and even modifying the layout of the designed cross-section. Whereas the two previous recommendations were implemented, the original design proved to be acceptable and there has been no need to reinforce the designed cross section throughout construction.

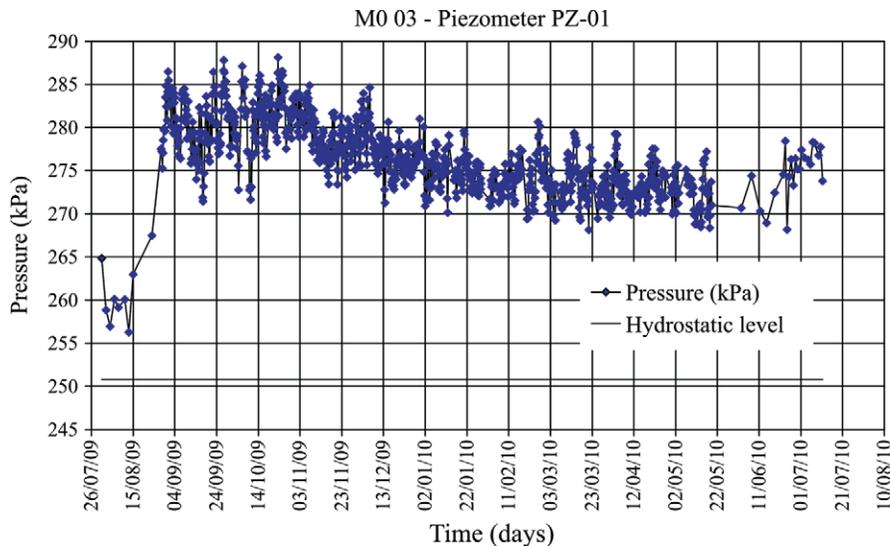


Figure 10 - Variation in pore water pressure with time.

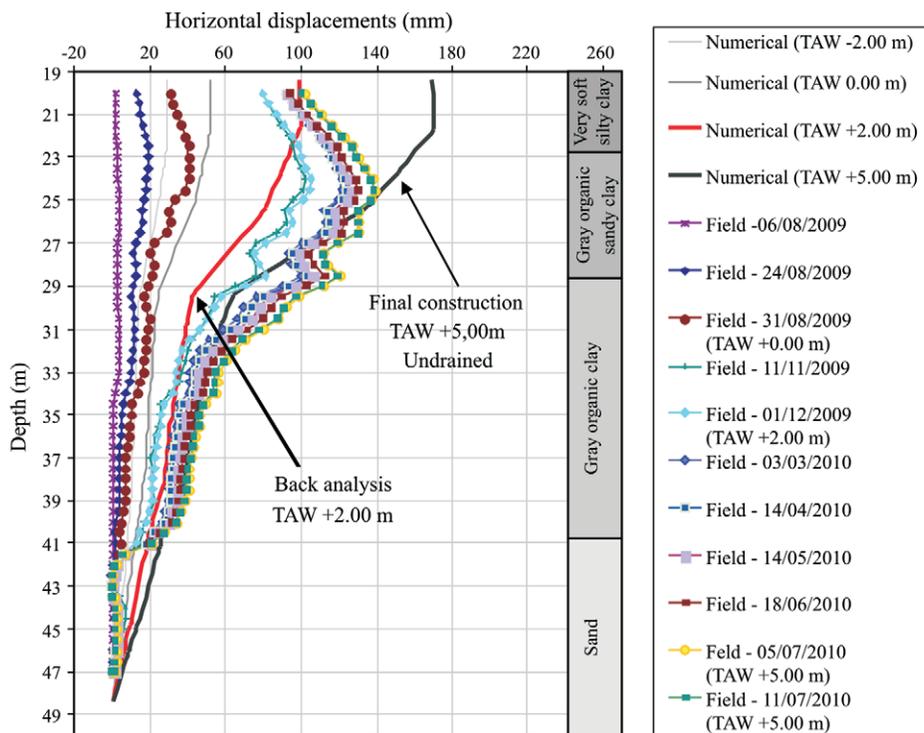


Figure 11 - Measured and predicted horizontal displacements.

Finally it is worth mentioning that last readings showed that the rates of displacements have decreased substantially and hence the settlement monitoring program was terminated in May 2012. Minor secondary settlements will occur in the future.

4. Numerical Simulation

Up to this point, the implementation of the observational method has been discussed on more empirical bases

and previous experience of the authors, i.e. selection of quantities to be observed during construction and comparisons to their anticipated values on the basis of the working hypothesis. However, in parallel to the described procedure as postulated by Peck (1969), an interactive analysis (this time with numerical tools) conceived to support the decision-making process was performed to evaluate the influence of partial drainage paths in the measured behavior. The numerical simulation was carried out in plane-strain

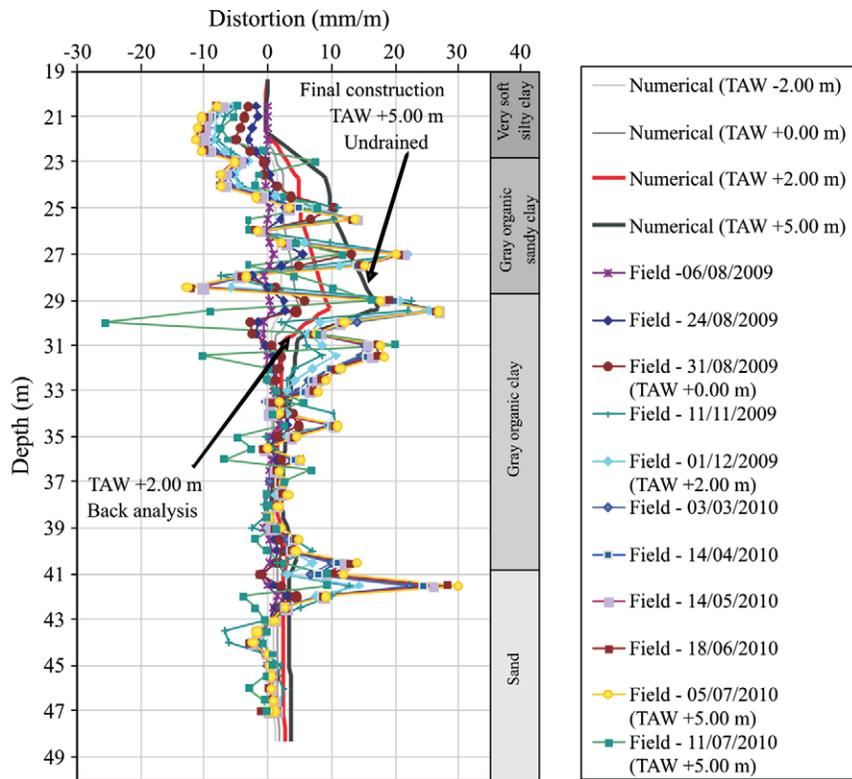


Figure 12 - Measured and predicted vertical deviation with depth.

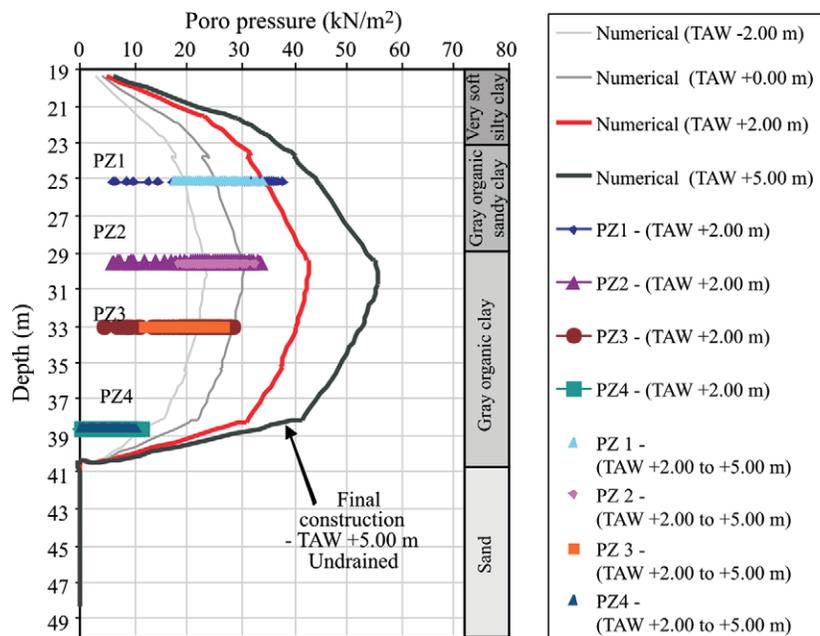


Figure 13 - Measured and predicted pore pressure with depth.

conditions using the modified Cam-Clay model, and the Biot theory for consolidation. After a throughout calibration of the program to local conditions (Dienstmann, 2011), displacements and excess pore water pressure dissipation during and after construction were predicted.

Finite element calculations enabled the back analysis of available measured data recorded from previous construction stages to be performed before predicting future embankment response. Numerical analysis becomes an interactive curve fitting process of displacement and pore

pressure *versus* time data, designed to refine the constitutive parameters on the basis of information generated in the interactive analysis cycle. Once the set of design parameters and boundary conditions are refined, a subsequent analysis is performed and used to guide the next stage of construction and final stability checks.

Figure 11 shows a plot of the computed horizontal displacements (elevation up to +2.0 m and to +5.0 m), using the finite element solution, along with the experimental data. The first analysis (or back analysis) at TAW +2.00 m was made as a prior evaluation of model and drained conditions, showing qualitative agreement with the experimental data, displaying a physical variation of lateral movements with depth that coincides with field measurements (maximum displacement around 100 mm). Since predicted displacements show promising overall agreement with experiment data, the analysis was extrapolated to predicting the behavior of the breakwater for elevation +5.0 m. Results presented in the same figure show field measurements on the final construction stage (TAW +5.00 m): maximum displacements at the seabed coincide with measured values (of the order of 140 mm) at the depth of about 25 m.

Similarly variations of vertical deviation and pore pressures with depth are presented in Figs. 12 and 13, respectively. Two construction phases are illustrated: back analysis at +2.00 TAW and final construction at +5.00 TAW. Although both vertical deviation and pore pressures show a general good agreement with the field performance, a close inspection of the data reveals that vertical deviations are underpredicted and pore pressures overpredicted.

The numerical work was particularly useful in demonstrating that there were no signs of failure in any of the construction stages. Additional work was essential to separate out the effects of drained and undrained loading on predicted and observed measurements of displacements. Consider the example illustrated in Fig. 14, in which vertical deviation is plotted against the rate of vertical deviation for measurements recorded at Station MO03 at the depth of 29.40 m. Numerical predictions for undrained loading up to failure are confronted to undrained loading followed by consolidation of the breakwater at an elevation of +5 m (predictions that correspond to observed field performance). In both cases the vertical deviation increases continuously to fairly high values of the order of 5% irrespectively to the drained path indicating that measures of vertical deviation alone cannot be adopted as risk analysis criterion. On the other hand, the rate of vertical deviation seems to be a good predictor of instability given the fact that it increases considerably during undrained loading and reduces during consolidation. The combined analysis of vertical deviation and rate of vertical deviation gives the best approach to risk assessment irrespectively to the need of cross correlating displacements to pore pressure measurements to depict signs of drainage.

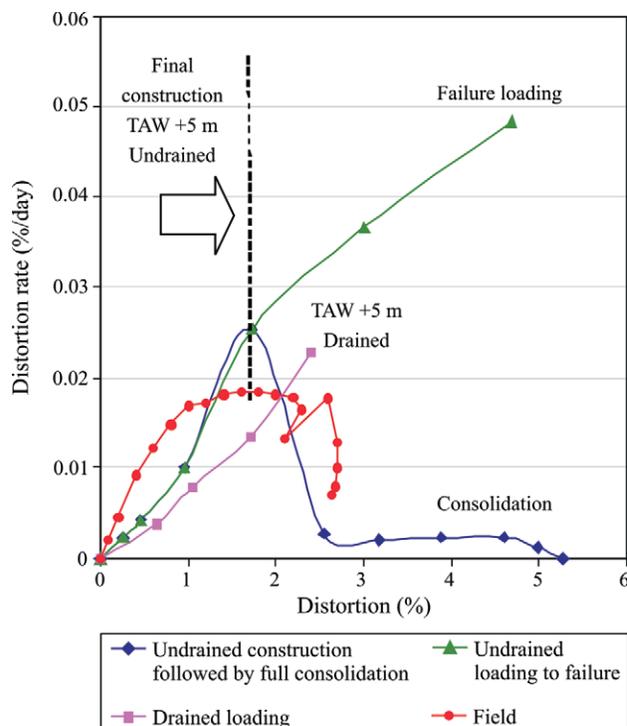


Figure 14 - Vertical deviation versus rate of vertical deviation at 29.38 m depth (Station MO03).

Finally is it worth emphasizing that a computational *risk assessment* approach for breakwaters cannot rely on *pre-established* reference acceptable limits of performance; acceptable limits have to be *adjusted* according to soil and loading conditions as well as the geometric representation of the structure to be constructed.

5. Discussion

From the years since Terzaghi and Peck first conceived and started working with the Observational Method to today, its use has been generalized, discussed in conferences and seminars, criticized, new names have been used to address it. But, has anything essential really changed from the early period?

It is the authors' opinion that important contribution for using the OM in an efficient and accurate way, in order to respond to today's societies demands, came with the technological developments introduced in monitoring the behavior of structures and earthworks, as well as the tremendous potential of analysis brought by numerical simulations. But, the essence behind the driving force to use the OM as a decision tool comes from Society, investigating, questioning and judging the performance of infrastructure works, in their interface with people and their products.

In all predictions of future behavior, even when based in back-analysis of data and parameters derived from previous stages of construction, the engineer is faced with an inevitable scatter of results derived from the best methods

available. This scatter may be minimized when a well planned site investigation, in tune with the geological-geomechanical model of the site, supports the elaboration of a sound and solid physical model of anticipated behaviors, which in turn helps choosing mathematical models to better represent them.

Determinism, and deterministically derived prediction behavior values, cannot be seen as a realistic tool when the variability of geomechanical and hydraulic parameters, the representativeness of constitutive models and of the numerical simulations are incorporated in the decision making process. Use of mathematical models that well represent the anticipated physical model transmits reliability to the risks management process, intrinsically linked with the

confirmation that the observed behaviour of the structure is within the anticipated range.

Table 2 shows a comparison between the general principles of application of the OM and the specific ways in which the method has been applied at the Rio Grande breakwater project. By doing so it is possible to summarize the key issues raised throughout the paper, highlighting the concepts that are deeply involved with the OM: that of robust design, safety case and contract flexibility. The British Institute of Structural Eng. (IStructE, 1990) and the Health and Safety Executive (HSE, 1996) both discuss the need of robustness in structures, which Burland (2006, 2008) defines as “the ability of absorbing damage without collapse”, which is broader than the concept of ductility, “the ability

Table 2 - Comparison of general conditions for application of the OM with the specificities of the Rio Grande Breakwater.

Conditions for application of the observational method		Application of the method in the rio grande breakwaters
Investigation	Sufficient geotechnical investigation must be available.	Boreholes, field tests (SPT, vane, undisturbed samples, piezocone) and laboratory tests. A horizontal continuous soft soil layer has been identified as the main object of attention.
Robustness	The system constituted by the structure and the terrain must be robust, meaning that it shall not be liable to an abrupt and unpredictable change from a stable to an unstable condition.	Soft soil is not highly sensitive and is in a normally consolidated condition during the most critical phase of construction. Therefore, ductile behavior.
Risk	Possible malfunction mechanisms must be identified and understood.	Identified risks: foundation shear failure during construction and excessive settlement after completion. Risk of acceleration and failure due to tertiary creep considered low because of low sensitivity of clay and of precedents in the Rio Grande area.
Monitoring	Instrumentation must cover all identified forms of risk materialization.	Horizontal displacements (inclinometers), vertical displacements in various depths (magnetometers) and pore pressures (piezometers) have been used.
Stages	Stage construction or velocity of construction must be smaller than the time required to interpret the monitoring data and to implement changes if necessary.	Rio Grande breakwaters constructed in four stages (-10 m, -5 m, +2 m and +5 m). In each stage, the increase of loading was gradual and could be interrupted.
Criteria	Criteria and procedures to confirm that the behavior is inside the anticipated range must be defined.	As far as the shear failure risk was concerned, the evolution of horizontal displacements and distortions based on limits from previous experience was the first criterion. Finite element analyses with parameters adjusted from one stage to the next were carried out in parallel. The long range settlements have been estimated from settlement plates installed in the top of the breakwaters.
Changes	Design must allow for changes in velocity of loading, intensity of load or construction procedures.	Loading could be stopped (as, in fact, happened once) and the design could be modified (which was not necessary)
Agility	Well defined decision hierarchy and management conditions to rapidly impose the changes considered necessary.	Direct contact between Consultant and Contractor with the proactive participation of the Owner allowed quick enforcement of decisions.
Flexibility	Contracts must be flexible in order to accommodate eventual changes in the construction period, loads or procedures.	Eventual interruptions in loading have been anticipated (and concretized in one occasion). If there was the need to change the design (which did not happen) contractual terms between the Contractor and the Owner would have to be renegotiated.

of undergo inelastic deformations without significant loss of strength". Robustness is usually also provided by identifying the hazards and risks and by checking that the design proposed is able to adequately withstand them.

Nicholson *et al.* (1999) postulate that "risk control measures must be an explicit part of the safety management system required by regulations", like the UK Construction Design and Management Regulations established by Health and Safety Commission (1994), and in this context define a safety case as "a systematic and, where possible, quantified demonstration that an installation or system meets specific safety criteria".

Sound judgment is required, and the careful analysis and interpretation of tendencies of behavior becomes a powerful tool. A previously defined hierarchy of decision taking at the jobsite is required to allow the rapid implementation of any action as shown by the monitoring program and its interpretation. For these reasons contracts must be flexible to accommodate the necessary changes in construction geometry, time and procedures.

6. Conclusions

A review of the early works on Observational Design as revealed through the extraordinary contributions from Terzaghi, Peck, Lambe, de Mello and others demonstrated the strength of the pioneering ideas embraced by conceptual Soil Mechanics, as well as their links to geotechnical engineering practice. These early concepts do not contrast with the outcome of some modern soil mechanics research, and yet the proposed framework of OM has not been entirely incorporated to current ground engineering projects. Consequences are that optimized cost-benefit design is not always achieved and identification of the hazards and risks are not entirely accounted for, especially in large structures subjected to adverse geo-environmental conditions.

The Rio Grande breakwater design and construction offered a unique opportunity for a critical appraisal of the OM approach where a complete application of the method was necessary given the extreme adverse geotechnical and hydrogeological conditions of the site. It comprised a comprehensive soil investigation, the setting of working hypothesis of soil behaviour described by a synthetic physical and mathematical model and a monitoring program devised to document the breakwater performance. Results have been confronted to acceptable limits of behaviour which triggered pre-established contingency actions conceived to ensure the successful completion of the work under the principles of the Observational Method.

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