A Few Lessons from Two Dams and an Excavation Instrumentation Projects

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SUMMARY

This paper is a contribution to the instrumentation design of geotechnical structures and particularly to tailings dams. It brings the Author's experience with a few case histories including Morro do Ouro and Fundao tailings dams, as well as an unique deep excavation in Rio de Janeiro.

Lessons learned from each case history are outlined in the paper. Morro do Ouro Dam showed the need in very deep inclinometers when a high dam is built on soft rocks, as is the case of many tailings dams in Brazil.

The lessons from Fundao Dam are overwhelming, as the dam was very poorly instrumented. There is a need of inclinometers, short time lag piezometers and vibration recorders on the dam, among several other instruments.

Vibration monitoring, frequently forgotten in dam instrumentation, has proven to be a useful tool to detect damage.

Finally, the third case history emphasizes the need of a numerical 3D model in order to be able to design the instrumentation programme and to set up alarm levels.

KEYWORDS

Tailings dams, instrumentation, dynamic monitoring, vibration analyses, Plaxis 3D analyses

1. INTRODUCTION

Many large structures in Brazil are poorly instrumented and analysed, which became clear in the case of the Fundão Dam failure that took place in November 2015 (Morgenstern et al, 2016),

In the wake of this disaster, the Brazilian National Department of Mining (DNPM in Portuguese) and Brazilian Standard Organisation (ABNT in Portuguese) issued new updated standards for inspection and monitoring of high-risk dams (DNPM, 2017 and ABNT NBR 13028). However, the Authors believe that these 2017 standards are still too vague and incomplete when dealing with instrumentation and monitoring of tailings dams. They leave the responsibility of the decision on how to instrument and monitor the dam to the Consultant or Designer.

The purpose of this article is, therefore, to present lessons learned from a few cases of dam instrumentation and monitoring from the last 20 years of the Authors' practice. The paper will emphasise what we believe to be a must in dam instrumentation and monitoring and should be incorporated into our standards.

2. MORRO DO OURO TAILINGS DAM

The first case history is a very large gold tailings dam: Morro do Ouro Dam, located in Paracatu, MG, Brazil, nowadays belonging to Kinross Mining. Ortigao et al (2013) described the dam and a trial embankment built on the tailings. The dam was constructed by the centreline method and founded on soft phyllite rocks. The embankment dam height reached 120 m and its length is about 4 km (Figure 1). In 2003-4, the dam height was being raised from 80 m to 100 m. Numerical analyses carried out by the designers at that time considered a rigid foundation (Figure 2).

Figure 1 Morro do Ouro Tailings Dam

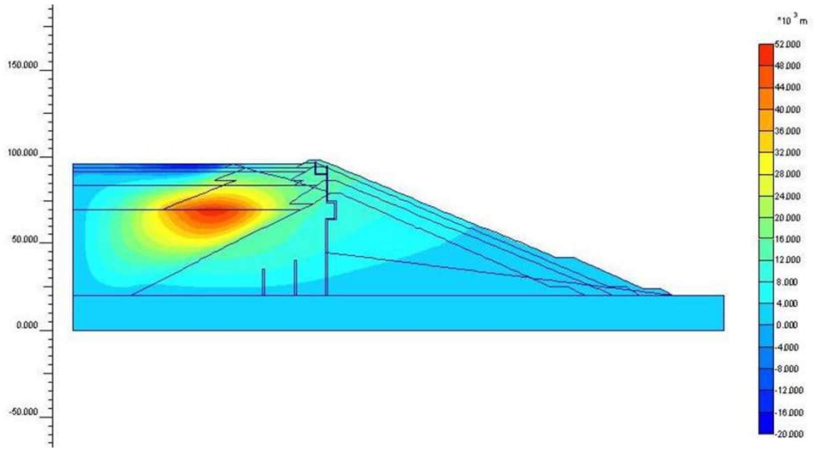


Figure 2 Plaxis 2D deformation analyses plot wrongly assuming rigid foundation

Then, in 2004, a few inclinometers were installed through the downstream slope and some of them were embedded 30 m in the soft rock (Figure 3). Readings (Figure 4) showed that the foundation was, by no means, rigid. Indeed horizontal deformation reached 80 mm. The soft rock under a large embankment loading lead to large deformation measured by the inclinometers. This led to a comprehensive investigation of the dam foundation characteristics and a review of the stability and dam safety.

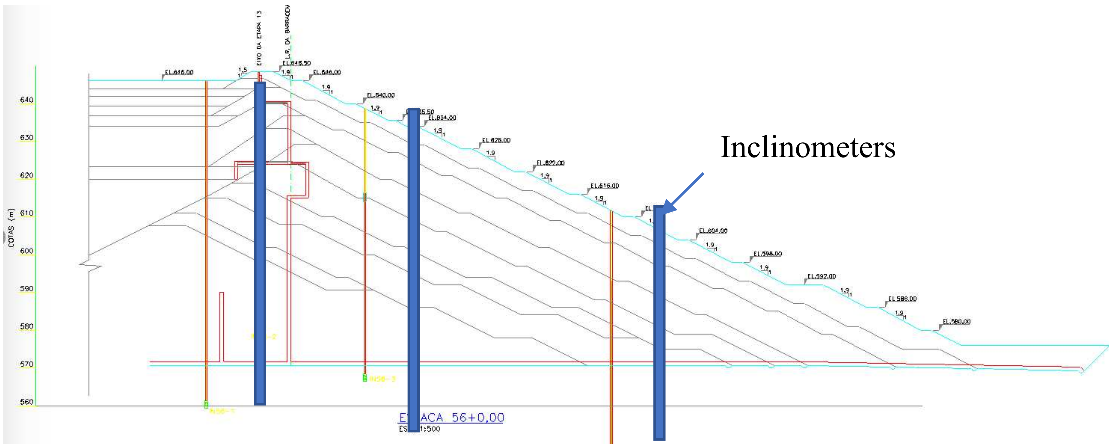


Figure 3 Inclinometer location Chainage. 56

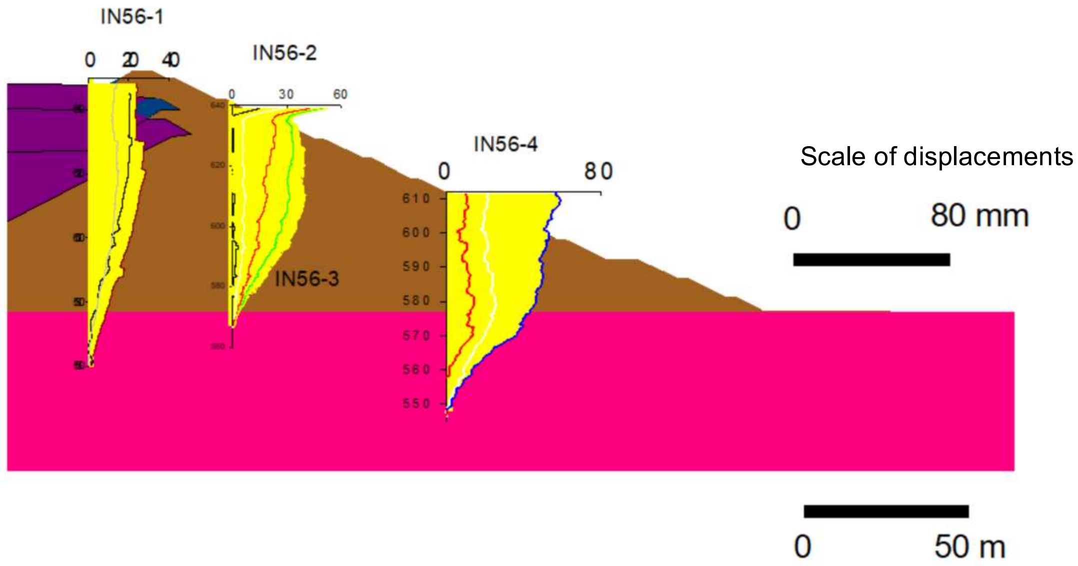


Figure 4 Summary of inclinometer measurements at Chainage 56

2.1 Lessons learned

High dams on soft rock foundation needs deformation measurements with inclinometers installed deep in the foundation. Reliable numerical models including the foundation can help to predict deformation behaviour and decide inclinometer depths. Advanced numerical calculations rely on sensitive parameters that must be obtained with correct site investigation. The foundation deformation must be taken into account when dealing with soft rock foundations.

3. FUNDÃO DAM FAILURE

The catastrophic disaster of Fundão Dam failure in 2015 has been extensively investigated by an panel of international consultants set up by Samarco Mining and headed by Professor Morgenstern (Morgenstern et al, 2016). The results were presented in 2016 and are available on the web (fundaoinvestigation.com/wp-content/uploads/general/PR/en/FinalReport.pdf).

The Authors collaborated with Samarco Mining well before the failure on a few projects, but not on Fundão Dam. One of these projects was a feasibility study of a dry tailings stockpile to be built on the existing tailings. This would provide tailings disposal volumes that would enable an additional 20 years mining operation time.

Back in 2014, the Authors carried out an investigation programme of dynamic and static liquefaction which included several seismic piezocone tests (CPTU) through the tailings. Figure 5 Location of CPTUs on the tailings, Fundao Dam (distances in metres). Figure 6 shows the CPTU location on Fundão Dam cross-section and Figure 6 shows a zoomed view and the water level position.

Figure 7 presents a typical CPTU log showing tip resistance, friction ratio, pore-pressures and SBT classification. This test was carried out down to 26 m depth, a small fraction of the tailings depth and shows interlayered loose sands and silts, typical of loose sandy tailings, and the fact that the water level is very high.

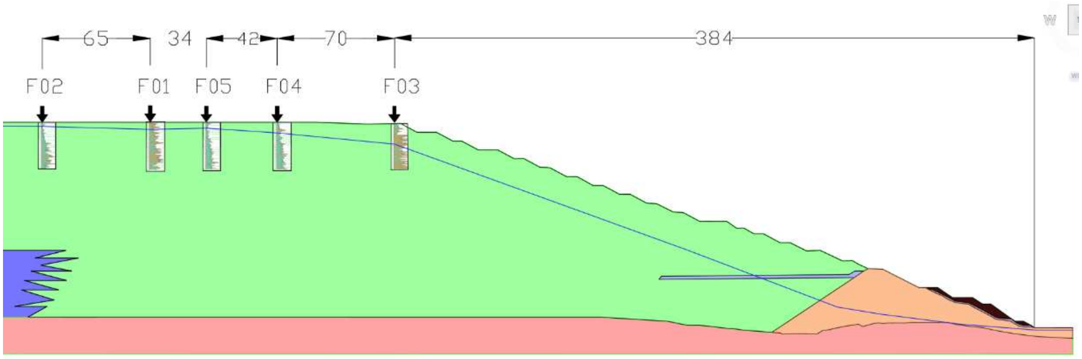


Figure 5 Location of CPTUs on the tailings, Fundao Dam (distances in metres)

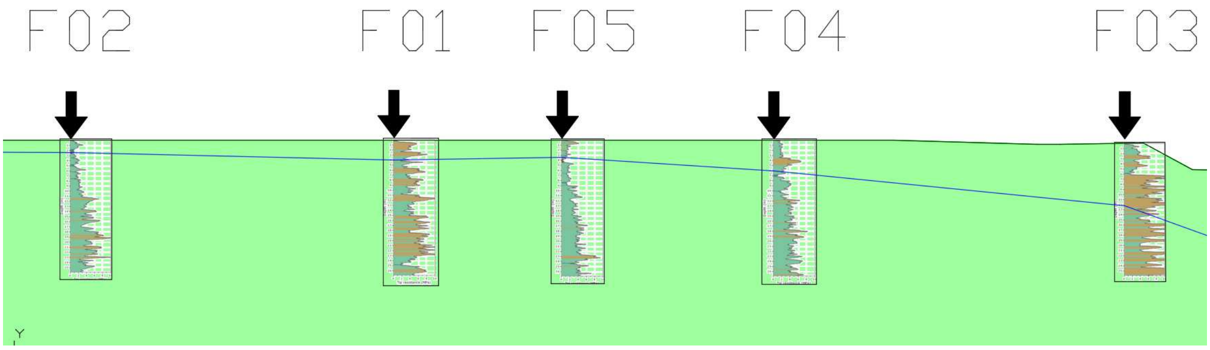


Figure 6   CPTU location on Fundão Dam crest and very high water level

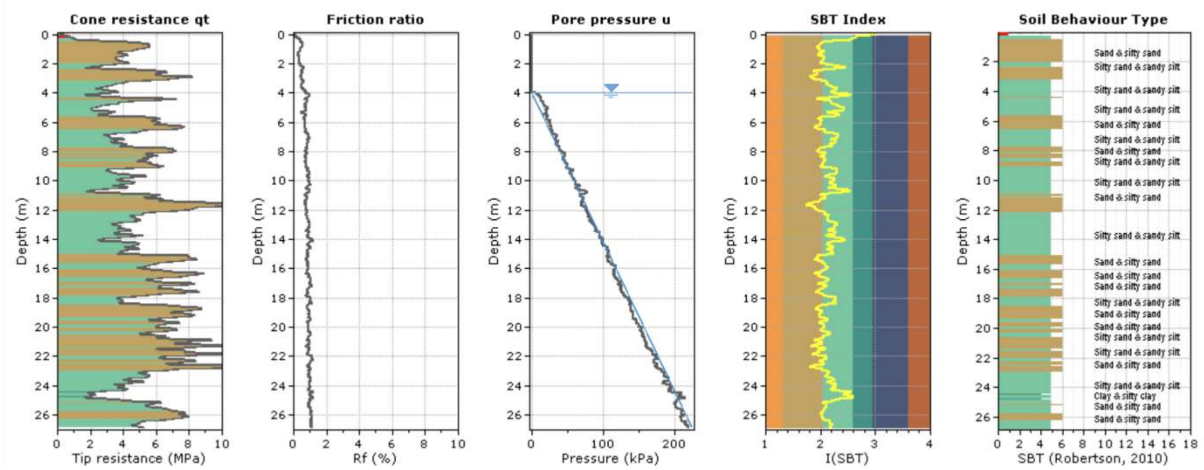
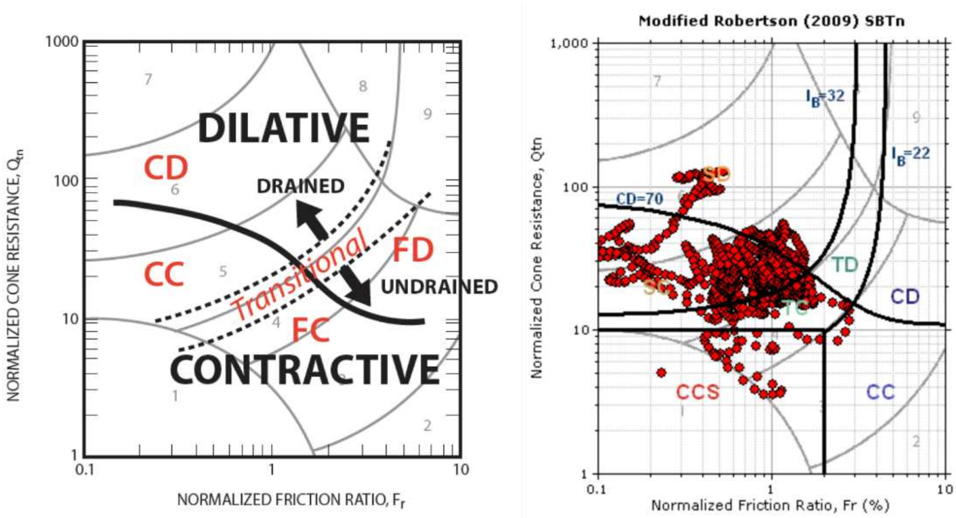


Figure 7   CPTU 01 log

The results were presented to Samarco's ITRB (International Technical Review Board) in November 2014, one year before the dam failure.

Robertson (2016) presented the plot in Figure 8 (normalized tip resistance versus normalized friction ration) which enables to differentiate dilative from contractive behaviour of soils through

CPT. Indeed, by plotting CPTU 1 in such a graph, most data points plot the lower zone where contractive behaviour takes place.



CD	Coarse-grained Dilative soil – predominately drained CPT
CC	Coarse-grained Contractive soil – predominately drained CPT
FD	Fine-grained Dilative soil – predominately undrained CPT
FC	Fine-grained Contractive – predominately undrained CPT

Figure 8 Analysis of CPTU 01 test data (Robertson, 2016)

Both the preliminary results (Figure 7) and the analyses carried out later (Figure 8) results indicated that two liquefaction precursors: high water level and loose and contractive sandy tailings were clearly present. These were two necessary conditions for static liquefaction to take place at the Fundão Dam failure, which occurred in November 2015.

The CPTU data gave very clear indication of high water level, but this information could not be cross-checked with very poor piezometer data from the dam instrumentation. Indeed, the main cause of failure was poor drainage. The bottom drainage close to the starter dyke was not working properly, and neither was the horizontal drain, depicted in Figure 5.

Morgenstern et al's (2016) report on the causes of failure explored extensively the triggering mechanism. One possible cause is dam horizontal displacement caused by disposal of fine tailings close to the starting dyke on the dam's left abutment. Morgenstern's report called it the *toothpaste effect* (Figure 9) in which a soft material is squeezed by vertical stress and force to displace horizontally. This could have triggered failure.

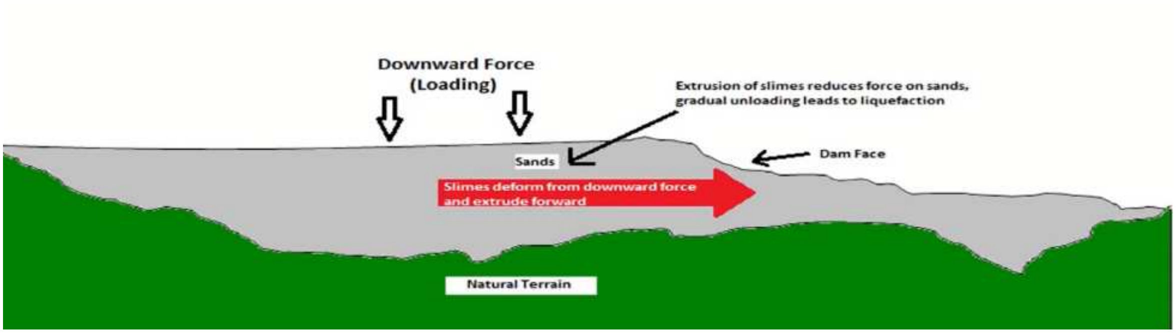


Figure 9 The toothpaste effect (Morgestern et al, 2016) leading to horizontal displacements and failure

Another possible cause is vibration or low-level seismic events that took place on the eve of failure. Indeed, people working on the dam crest felt these vibrations. Figure 10shows their epicentres, one of them located under Fundão Dam. This phenomenon may have led to an increase in pore pressures in the tailings, but this information cannot be checked, as there are no quick response piezometers, only open standpipe type and observation wells.

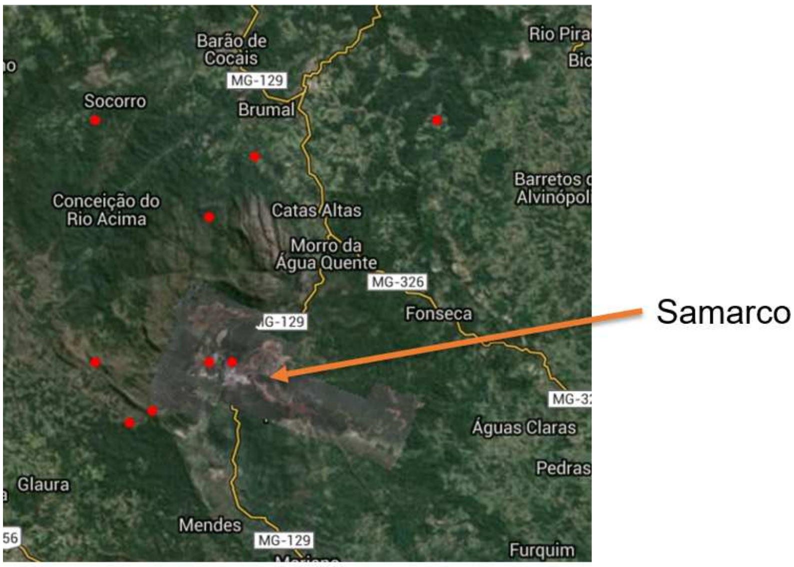


Figure 10 Epicentres of the seismic events taking place the eve of Fundão Dam failure

Questions that remain unanswered due to lack of good quality instrumentation: What was the effect of the vibrations on the pore-pressures? How much horizontal displacement took place in the dam, as reported by Morgenstern?

3.1 Lessons learned

1. Need for good high-quality piezometric data, including short time-lag piezometers capable of indicating any quick change in water pressure conditions;
2. Automated instrumentation systems are a must.
3. Vibration measurements
4. Inclinometers through the foundation.

Figure 11 shows the layout and type of instrumentation that the Authors would advocate for addressing the issues raised in Fundao Dam.

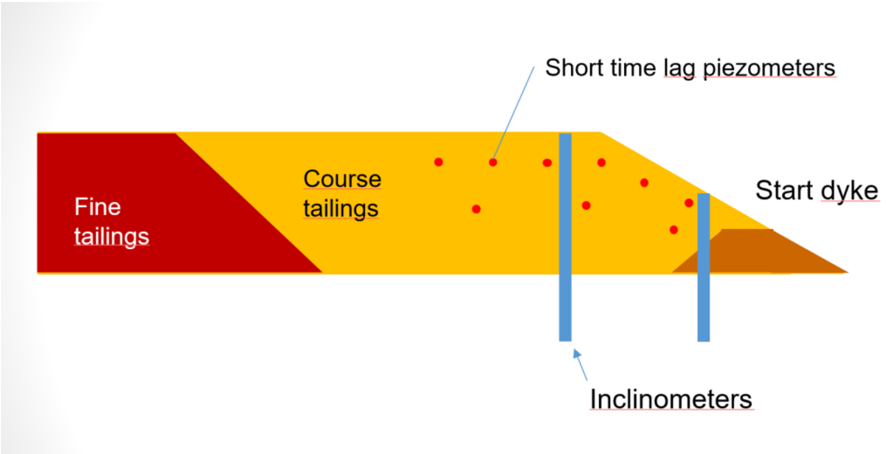


Figure 11 Instrumentation lessons from Fundão Dam failure

4. DYNAMIC MONITORING AND DAMAGE DETECTION

In the wake of the Fundão failure, concerns were raised in this country with regard to dynamic or seismic monitoring.

Assumpção et al (2016) analysed recent seismic events after the Fundão failure and showed that the Brazilian Standard ABNT NBR 15421 needs to be updated. Figure 12 presents PGA values (peak ground accelerations) recommended by current Brazilian standards and by these authors. The maps show that the centre of the country presents a large difference and the PGA values increased as compared to the current standard.

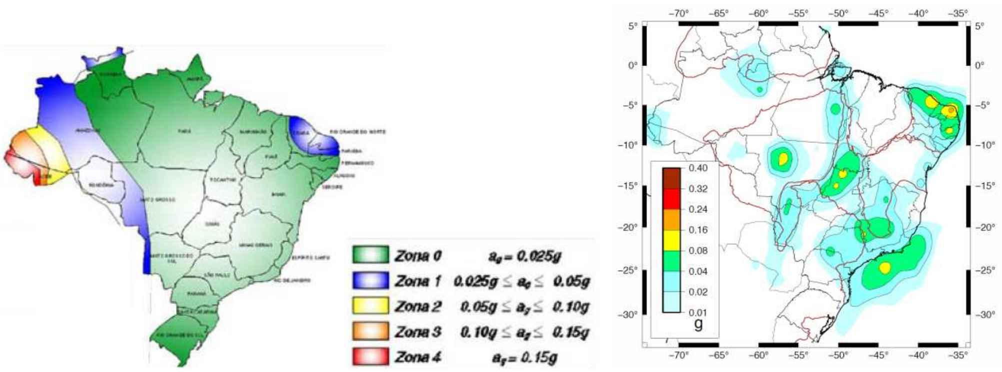


Figure 12 Recommended PGA (peak ground accelerations) for 2% probability of being exceeded and 50 years return period. Left: ABNT NBR 15421 recommendations, right Assumpção et al's (2016) recommendations

They conclude that monitoring should be continuous and our seismic hazards maps frequently updated.

In seismic areas, it is common to install accelerometers on the dam crest, in the foundation and on a rock outcrop nearby. This enables damage detection in case of a seismic event.

The Fundão failure has led to an increase in seismological network stations. However, seismologists usually do not take into account that vibration monitoring of dams enables damage detection at an early stage well before generalised damage takes place. The most common methods for carrying out damage detection (Jeary, 1997) are through spectral and damping analyses. This section explores spectral analyses only.

Indeed, most text-book dynamics show that in a simple undamped mass-spring system, the resonance frequency (f_r) is given by:

$$f_r = \frac{1}{2\pi}\sqrt{\frac{K_r}{m_r}} \qquad (1)$$

Where:

K_r is the modal stiffness.
m_r is the modal mass, i.e., the mass that interferes in this mode shape.
This explains that changes in frequency are related to changes in the stiffness of structures.

The following figures show an example of a structure affected by vibrations due to pile driving in the vicinity. Figure 13 presents dynamic measurements on the top of the structure by means of an accelerometer. When pile driving started, the plot shows very large accelerations which reached 0.2g. Velocities (Figure 14) reached values of 10 mm/s, which is about half of the DIN 4150 limit. Therefore, according to this standard, it would not have damaged an industrial or commercial building. Nevertheless, the following paragraphs tell a different story.

Figure 15 shows the spectrum analysis, or power spectrum density (PSD) as a function of frequency for baseline measurements, *i.e.*, before pile driving started. It shows a fundamental resonance at approximately 5 Hz. However, after pile driving started, the first resonance decreased to half of this value, as low as 2.5 Hz (Figure 16). This is a clear indication that the structure was damaged by excessive vibrations, reduced frequency (f_r) leads to a reduction in stiffness, according to equation (1).

This kind of instrumentation and analysis is quite cheap and should, in the Author's view, be part of dam instrumentation.

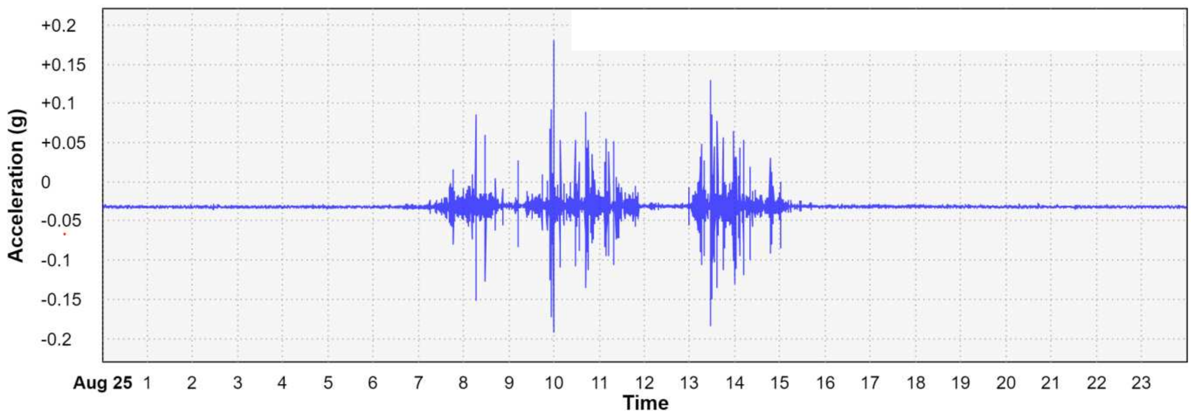


Figure 13 Example of dynamic measurements on a structure affected by pile driving nearby

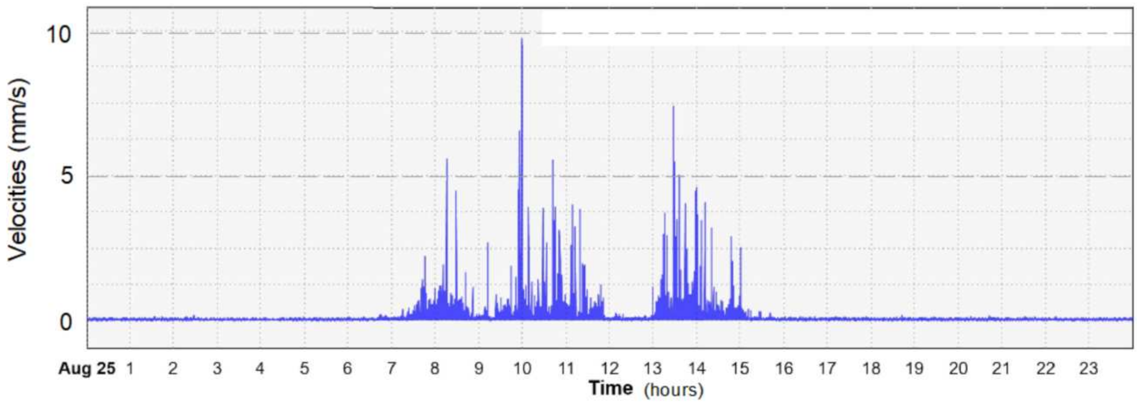


Figure 14 Velocities versus time

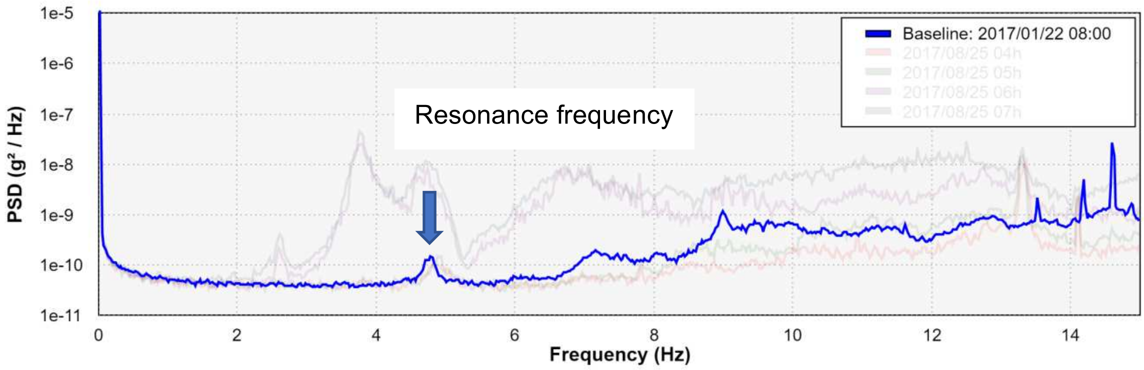


Figure 15 Baseline spectral response

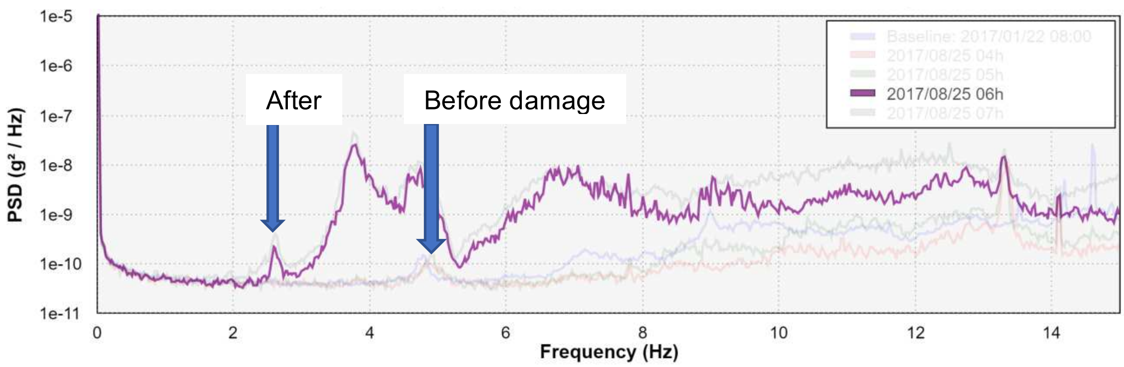


Figure 16 Spectrum response during pile driving showing a large reduction in resonance frequency

5. DEEP EXCAVATION FOR WATER RESERVOIRS IN RIO DE JANEIRO

The World Cup and Olympic Games that took place in Rio de Janeiro in 2014 and 2016 brought a considerable amount of investments in public works. Duarte et al (2013) describe works to control flash floods around Maracanã Football Stadium which hosted several games and the Olympic closing ceremony. The flash flood works control works included 3 deep reservoirs. This section describes the challenge for the design and construction of Varnhagen Square water reservoir.

It was located at the Varnhagen Square which has a triangular shape (Figure 17) and therefore, imposed geometrical constraints to the project. After analysing various design scenarios, the Author's proposed a unique "Mickey Mouse" design of a central circle with 45 m in diameter and two intersecting "Mickey Mouse ears" with 30 m and 24 m in diameter respectively.

Figure 17 Varnhagen Square reservoir. Left: top view and boundaries;Right: top view of the "Mickey Mouse"reservoir

The design employed an 800 mm thick diaphragm wall, 2.8 m wide panels or sections, wall taken to the bedrock top, approximately 20 m below ground level, leading to a total volume of 45 800 m^3.

Site investigation consisted of several drillholes through soils and bedrock, pressure meter tests (PMT), in-situ permeability, geophysics and dewatering pumping test. Figure 18 presents a summary, which includes hard soils with *N*(SPT) values above 20 of interlayered hard silty clays and sands overlying gneiss. The water level was about 2 m below ground level.

The excavation design had a few key aspects which made it feasible and safe. Firstly, the water level could not be affected in the excavation vicinity in order to avoid ground settlements that could affect the surrounding buildings. This was ensured by taking the diaphragm wall to the bedrock and carrying inside, before excavation, a pumping test, which showed the water level outside was not affected. Jet-grouting (JG) columns, 60 cm in diameter, were installed at the joints to close the gap between diaphragm wall panels to ensure sealing against water leakage.

The second key aspect was how to build the Y joints between diaphragm wall panels (Figure 19) which are submitted to high compression forces. The selected design was to improve the ground around the Y intersections with 1.2 m diameter, 1 m centre to centre, JG columns. This enabled the intersection concrete to be cast during excavation.

Finally, diaphragm wall panels present a flexible hinge between panels. To hold them in place, the design adopted a series of concrete beams or bracing "rings" to hold them in place (Figure 20).

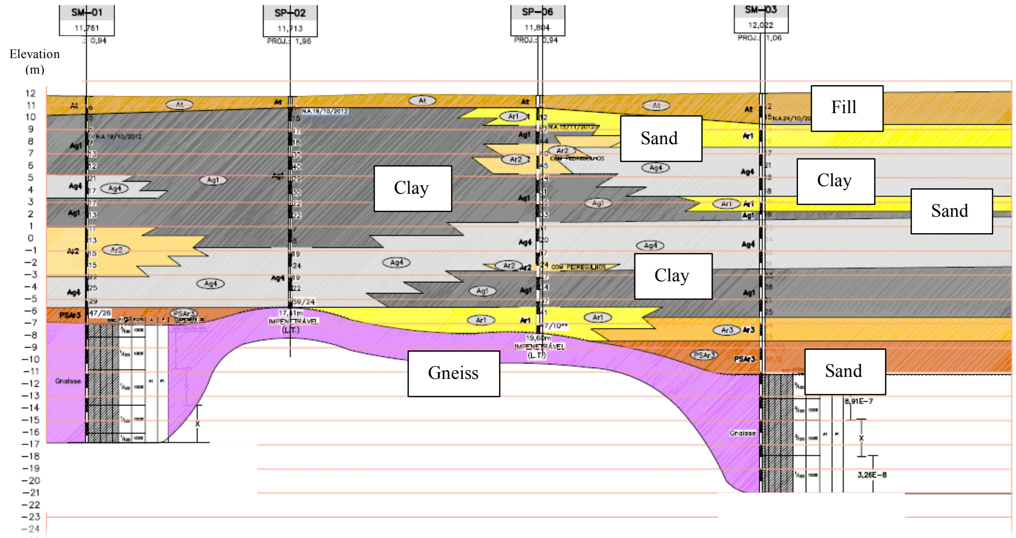


Figure 18 Ground profile

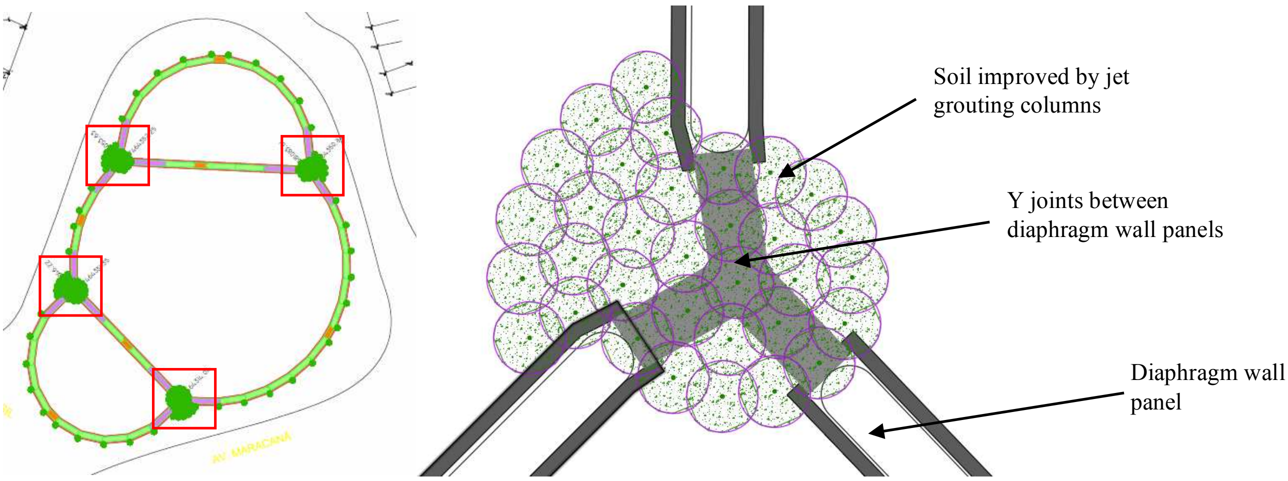


Figure 19 Plan view of the excavation showing the Y nodes between intersecting diaphragm wall panels. Left: location of Y node; right: details of the ground improvement with JG columns

Figure 20 Varnhagen Square "Mickey Mouse" reservoir during excavation showing bracing rings

5.1 Plaxis 3D Numerical Model

A complex structure like this excavation requires a 3D model. Figure 21 shows the adopted geometry and Table 2 present selected parameters.

The model used the HSM (Hardening soil model) constitutive model to represent soils and Hoek Brown for the bedrock. Soil's Young moduli were figured out from PMT tests, through numerical simulation using the same Plaxis constitutive model.

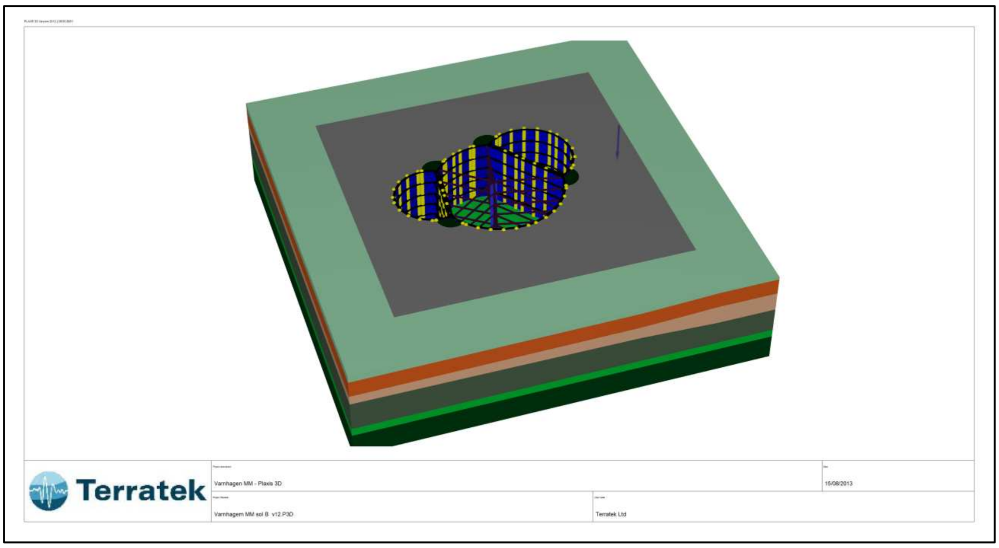


Figure 21 Plaxis 3D numerical model

Table 1 Selected soil parameters for Plaxis HSM model

Property	**Symbol**	**Unit**	**Embankment**	**Sandy clay**	**Silty clay**	**Residual soil**
Unit weight	γ	kN/m³	19	19	19	19
Secant modulus at 50% stress level	E_{50}	MPa	11	33	55	77
Oedometric modulus	$E_{oed} = M$	MPa	11	33	55	77
Unloading-reloading stiffness	E_{ur}	MPa	30	90	150	210
Drainage type	-	-	Drained	Drained	Drained	Drained
Undrained strength	c_u	kPa	-	-	-	-
Cohesion	c	kPa	10	22	40	20
Friction angle	ϕ	[º]	30	30	33	35
Dilatancy angle	Ψ	[º]	0	0	1	2
Janbu Coefficient	m	-	0.6	0.8	0.8	0.6
Reference stress for stiffness	p'_{ref}	kPa	6	101	191	300

Table 2 Selected bedrock parameters

Property	**Symbol**	**Unit**	**Soft rock**	**Intact rock**
Unit weight	γ	kN/m³	25	
Young's Modulus	E	MPa	200	
Poisson ratio	μ	-	0,25	
Intact rock uniaxial compressive strength	σ_c	MPa	250	
GSI	*GSI*	-	50	90
mi	*mi*	-	28	
D	*D*	-	0.8	0.3

5.2 Plaxis 3D Results

The following selected figures present displacements, moments and forces on structural elements. The figures in this paper represent less than 5% of the figures generated by the model, The structural design of these elements used this information.

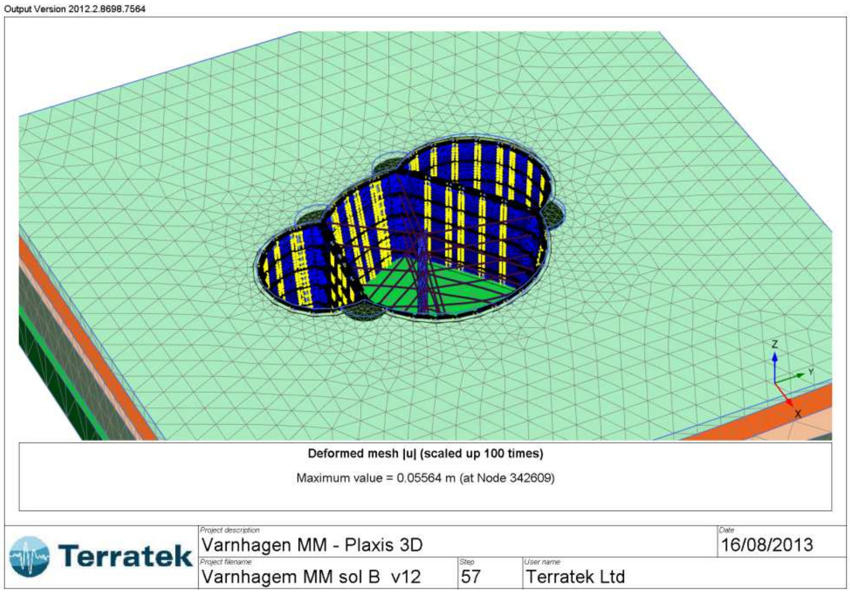


Figure 22 Deformed mesh

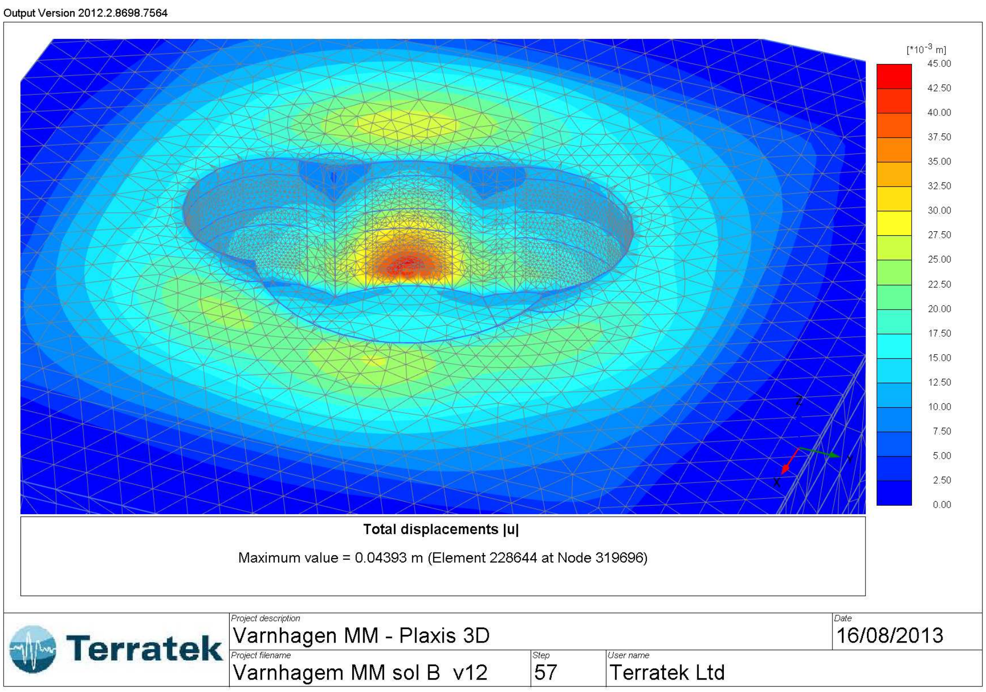


Figure 23 Total displacements, u

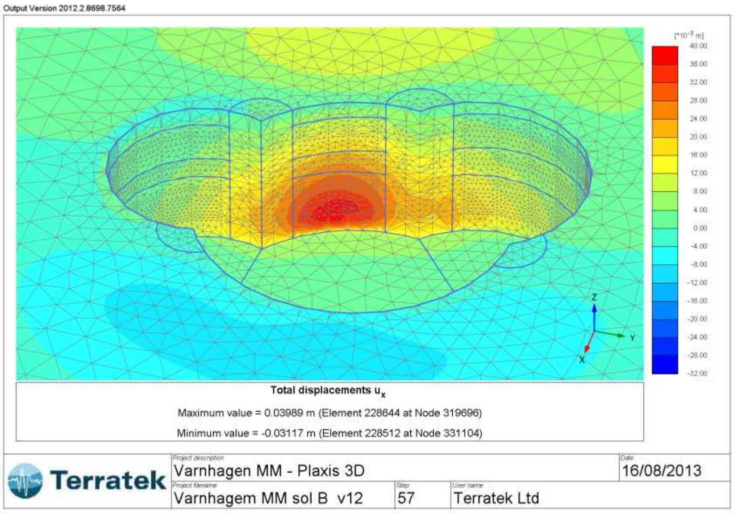


Figure 24 Total displacements, u_x

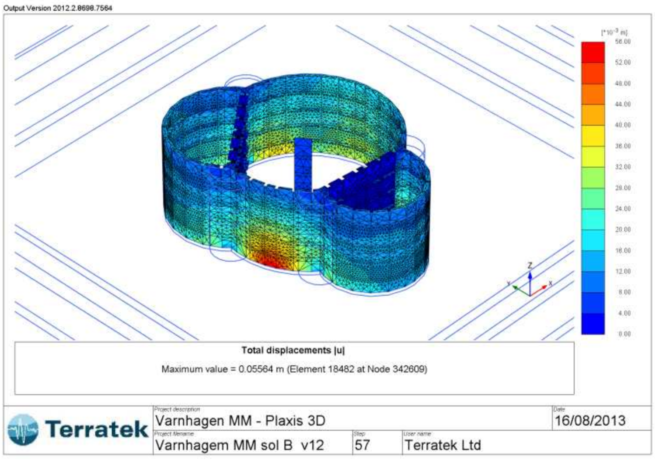


Figure 25 Total displacements, u

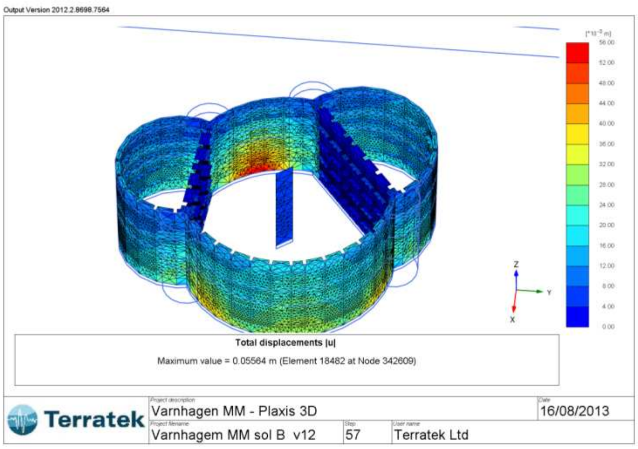


Figure 26 Total displacements, u

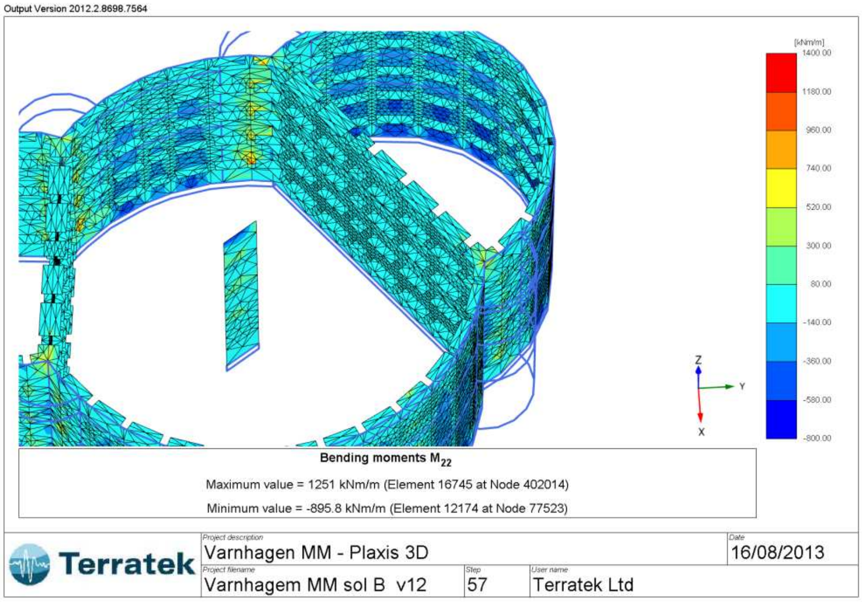


Figure 27 Bending moments

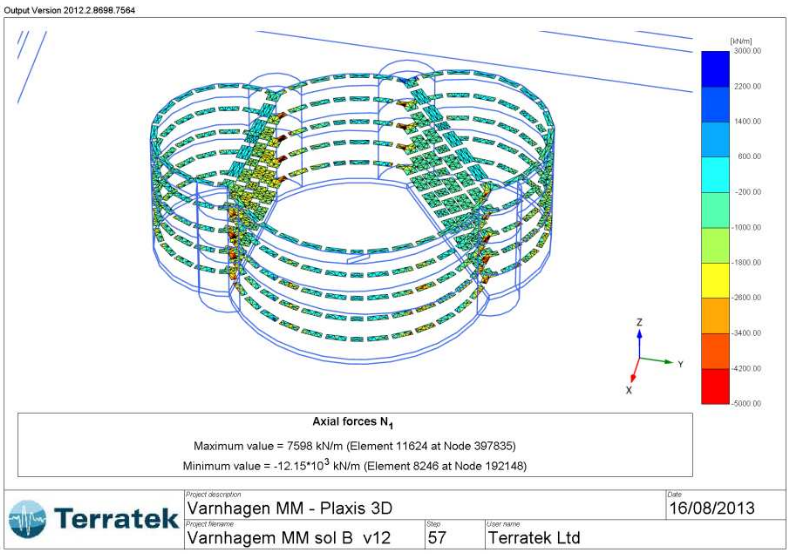


Figure 28 Axial forces on the bracing rings

5.3 Instrumentation and monitoring programme

Figure 29 presents the instrumentation plans with seven inclinometers, many piezometers and all precise levelling of settlement pins installed in all buildings in the vicinity.

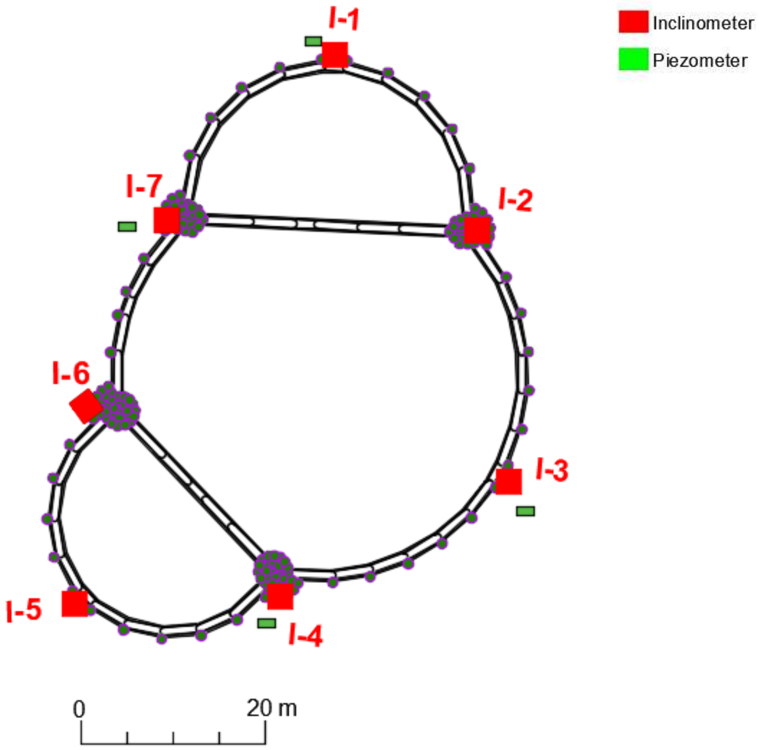


Figure 29 Instrumentation plan

Instrumentation results indicated that the water level outside the excavation did not change and the surrounding buildings did not show any significant settlement. The only significant observed effect was horizontal displacements around the excavation.

Figure 30 shows a comparison between predicted and measured horizontal displacements in three inclinometers, out of the seven which were monitored. The I-1 instrument is located on the north wall (Figure 29). It shows very small displacements, less than 10 mm and less than predicted. Inclinometers I-6 and I-7, located at the West Y joints, have shown displacements of about 30 mm, larger than predicted, but still less than 0.2% of the excavation depth.

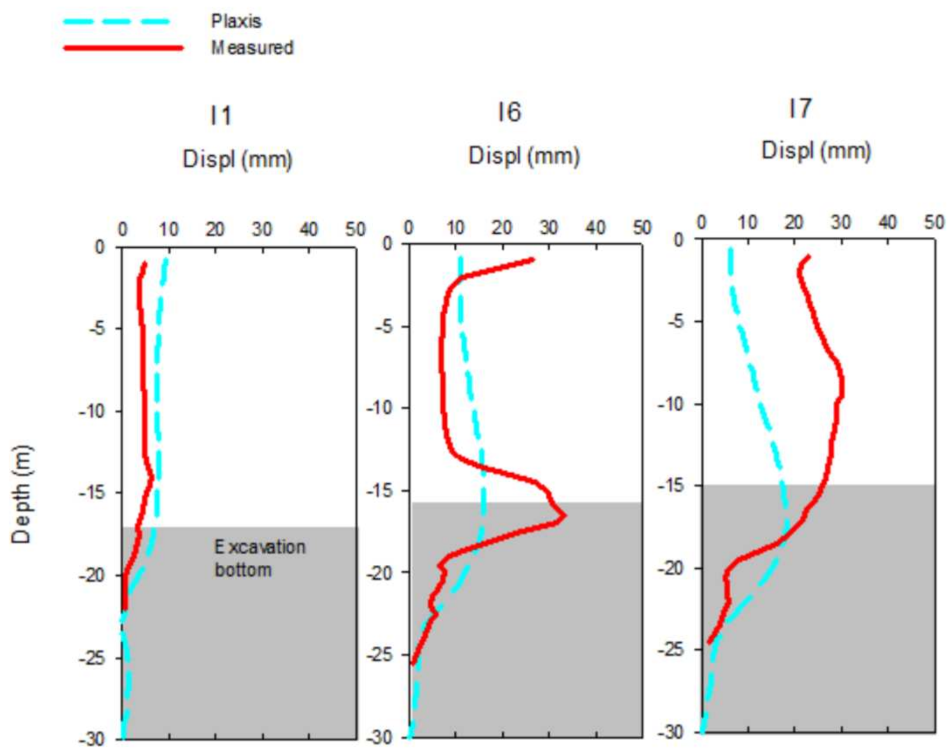


Figure 30 Comparison between predicted and measured horizontal displacements at inclinometer I-1, I-6 and I-7 (I-1 at the north side, I-6 and I-7 close to the west Y joints)

5.4 Lessons learned

3D models are very important to predict the structural behaviour and to enable a proper design of an instrumentation system in a complex geometry configuration. Models like these must rely on a comprehensive site investigation programme including CPT, PMT, seismic and other in-situ and laboratory tests.

During excavation the monitoring results have shown good behaviour for the wall deformation and this led to additional construction savings, as possible struts which were initially considered to limit horizontal deformation proved not to be needed.

6. CONCLUSIONS

The main conclusions from the case histories presented herein are:

- Large dams on soft rocks do need inclinometers installed deep in the dam foundation, reaching a depth beyond the displacement field caused by the dam;
- Fast response piezometers, such as the vibrating wire type, are a must alongside open standpipe piezometers and observation wells;
- Automatic monitoring should be implemented;
- Vibration recorders should be a must on the dam crest. They are relatively cheap, easy to install and enable prediction of approaching damage;

- Monitoring and analyses of complex structures should include 3D numerical models that should be based on comprehensive site investigation and analyses.

7. ACKNOWLEDGEMENTS

Dr A Jeary has guided the first author through the world of dynamics since they first met in the early 80's at the BRE Building Research Establishment, Watford, England. Dr Jeary provided the example of the dynamic monitoring and damage detection.

Dr P K Robertson was consultant to Samarco during the 2014 investigation and supported data analyses of field testing programme.

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