Contribution of geotechnical engineering in the rehabilitation of buildings and infrastructures

Contribution de la géotechnique a la réhabilitation des bâtiments et infrastructures

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ABSTRACT

The infrastructure of a country is the valuable asset which ensures its social and economic growth. However, ageing effects, operational reasons, changes in codes and standards of safety, inadequate past design practices and construction imperfections, natural disasters and war acts, as well as the recent tendency to reuse abandoned infrastructures impose their rehabilitation. In this paper examples of the contribution of geotechnical engineering for the rehabilitation of infrastructures, mainly in urban environment, are shown.

RÉSUMÉ

L'infrastructure d'un pays est la fortune la plus précieuse qui assure l'évolution sociale et économique d'un pays. Pourtant, des effets dus au déroulement du temps, des raisons opérationnelles, des changements aux codes et aux règlements de sécurité, des pratiques des projets passées et inadéquates et des imperfections des constructions, des désastres naturels et des actes de guerre, en même temps que la tendance récente de réutiliser des infrastructures abandonnées imposent leur réhabilitation. Cet article présente certains exemples de la contribution de la géotechnique à la réhabilitation des infrastructures, surtout en milieu urbain.

Keywords: geotechnical engineering, infrastructure, monuments, rehabilitation, restoration

1 INTRODUCTION

"The quality of a nation's infrastructure is a critical index of its economic vitality. Reliable transportation, clean water, and safe disposal of wastes are basic elements of civilized society and a productive economy. Their absence or failure introduces an intolerable dimension of risk and hardship to everyday life, and a major obstacle to growth and competitiveness … The infrastructure must rank high among our priorities. We must ensure that our highways and subways can move us swiftly and safely; that our homes, farms, and industries are supplied with ample clean water; that we reduce and safely dispose of the increasing volume of poisonous wastes our society generates; and that we provide the structural underpinning for a robust and competitive economy" (National Council on Public Works Improvement, 1988).

In 1889, 2,209 lives were lost when the South Fork Dam failed above Johnstown, Pennsylvania. The 1928 St. Francis Dam failure killed 450 persons. During the 1970s, the failures of the Buffalo Creek Dam in West Virginia, Teton Dam in Idaho and the Toccoa Falls Dam in Georgia collectively cost 175 lives and more than $1 billion in losses. Seven people were killed in Kauai, Hawaii on March 2006 when an earth dam failed, unleashing nearly 300 million gallons of water, smashing down trees and sweeping away homes. The failures of Silver Lake Dam in Michigan in 2003 ($100 million in damages and economic losses of $1 million per day) and the Big Bay Lake Dam in Mississippi in March 2004 (100 homes destroyed) are current reminders of the potential consequences of unsafe dams.

At 9:15 am on Friday, October 21, 1966 a waste tip slid down a mountainside into the mining village of Aberfan, near Merthyr Tydfil in South Wales. It first destroyed a farm cottage in its path, killing all the occupants. The slide engulfed the Pantglas Junior School, just below (Figure 1), and about 20 houses in the village before coming to rest. 144 people died in the Aberfan disaster: 116 of them were school children.

A massive landslide occurred in the Las Colinas neighbourhood of Santa Tecla, El Salvador, Central America as a result of the M = 7.6 earthquake of January 13, 2001. The landslide buried many houses

Figure 1. The Pantglas Junior School, Aberfan, South Wales engulfed by the debris of the flowslide.

Figure 2. Oblique aerial view of landslide that buried Colonia Las Colinas, El Salvador.

in the neighbourhood under tons of earth (Figure 2), leaving as many as 1,200 people dead.

A clogged sewer line caused a sink hole in Portland, Oregon, USA on December 2006, which swallowed a truck – which had been sent to clean the sewer. Natural gas and water lines were also ruptured in the accident. A similar accident happened in Guatemala City, Guatemala on 22nd February 2007. A hole of 30.5 m diameter and 49 m depth opened (Figure 3), swallowing five houses and killing 3 persons. The accident was the result of the leakage of a deep buried sewer pipe in tuffs (pyroclastic deposits).

A 120-ton concrete beam collapsed onto Interstate 70 in Washington County, Pennsylvania, USA, barely missing passing motorists on December 2005, while recently (2nd August 2007) the catastrophic collapse of the I-35W highway bridge over the Mississippi river in Minneapolis, Minnesota, USA, left 5 dead persons and 111 injured (Figure 4).

The above examples of infrastructures failures show clearly the need for rehabilitating ageing and imperfect structures.

Figure 3. Leakage of a sewer pipe created a hole of 30.5 m diameter and 49 m depth in Guatemala City.

Figure 4. Aerial view of the collapsed bridge in Minneapolis (2nd August 2007).

Furthermore, the redevelopment and reconstruction of urban areas are essential to ensure economic development. Since the abandonment of an engineering structure, approaches to geotechnical design methods have changed. This, combined with added client requirements and changes to loadings (especially those of the earthquakes, which are of prime interest for earthquake prone countries), is another strong reason for rehabilitating infrastructures.

Rehabilitation of old structures, particularly in urban areas, often means considering existing foundations, bridges and earthworks for reuse, and it is important to understand the impact of these old structures will have on modern geotechnical design. Existing foundations might perform satisfactorily under new loads, even though they are decades old. However, they may not satisfy current regulations.

It is common practice that in order to provide safe continuing service, engineering structures require ongoing maintenance, monitoring, and frequent inspections, to assure their safety. Moreover, based on the aforementioned examples, after some years in use, their safety is assured by their rehabilitation, which is required because of:

- Ageing
- Operational reasons (increasing demands)
- Changes in codes and standards of safety (predicted increase in extreme events)
- Inadequate past design practices and construction imperfections
- Natural disasters (earthquakes, tsunamis, hurricanes-typhoons, landslides etc.)
- War acts

The "rehabilitation" of ancient monuments could be classified in the first case, however not with the meaning that all the monuments would be again usable – habitable, but that they will be restored to a certain level which is showing off the monument and safeguarding it and its visitors.

In the latter two cases the rehabilitation required could be as large as of the scale of a city, like rehabilitation of Bam City in Iran, after the devastating earthquake of 26th December 2003, and rehabilitation of Banda Aceh in Indonesia, after the catastrophic tsunami of 26th December 2004, or of the scale of a whole country, like rehabilitation – reconstruction of Iraq in order to improve and repair the infrastructure of the country in the aftermath of the 2003 invasion.

In this paper examples of the contribution of geotechnical engineering for the rehabilitation of infrastructures will be shown, some from the authors' experience, some from the literature.

2 GEOTECHNICAL INVESTIGATIONS FOR THE REHABILITATION OF INFRASTRUCTURES

The practical implications of good ground condition knowledge are obvious. It can save millions of euros through informed decisions about remedial works.

Ground investigation on rehabilitation sites often means considering existing foundations, resulting in too congested ground. Investigations may involve some form of drilling adjacent to, or through, existing foundations. Poulos (2005) mentions that both forms of drilling in pile foundations may have deleterious effects on the piles being investigated. The drilling of holes adjacent to piles will generally cause vertical and lateral ground movements and these will act upon the nearby piles, inducing additional stresses and movements. These effects may be particularly severe if the ground is highly stressed. Coring through the pile itself may cause loosening or even piping of the soil beneath the pile toe when "breakthrough" is achieved and the underlying soil is soft or loose. Also, SPT testing of the soil below the pile base may cause further disturbance if it is not carried out carefully, and the SPT rods are withdrawn too quickly, thus causing suction within the soil surrounding the hole. Thus the investigation process itself may help to accentuate the problem being investigated.

Rehabilitation projects such as these are likely to see a raft of sophisticated testing techniques coming into more general use. These include the continued evolution of smarter in-situ tests, such as the use of driven pressure meters and falling weight deflectometers obtaining stiffness parameters at swallow depths, as well as other geophysical equipment as the ground penetrating radar.

In the process of the rehabilitation of the Church of Saint Peter of the Dominicans (Agios Petros Dominicanon), at Iraklion, Crete (14th-15th century building), the foundation conditions of the church were investigated in order to give reasoning to the differential settlements and the consecutive cracks, which have appeared in the monument. The investigations consisted of three boreholes, with SPT tests on site and laboratory tests, later on, for the definition of the foundation conditions and a detailed geophysical investigation, involving seismic / acoustic tomography, for defining the details of the foundation bodies of the structure. The results of the geophysical investigation revealed that the foundation bodies of the church are resting on the bedrock, however there are some sections of the walls where either there in no foundation body, or the foundation body is resting on top of soft deposits (see Figure 5).

3 REHABILITATION OF BUILDINGS

The rehabilitation of buildings, from the geotechnical engineering point of view, refers, in the most cases, to the rehabilitation of their foundations. The main reasons for which this rehabilitation is required are either the uneven settlements, which the buildings may have presented, or the low bearing capacity of the foundations, compared to the loads which will be applied.

According to Poulos (2005), the methods for correcting the uneven settlements of buildings' foundations can be divided broadly into two categories:

i. "Hard" methods, which rely on the application of some form of direct force to the building, like:
    - Application of force by anchor stressing
    - Application of additional loading

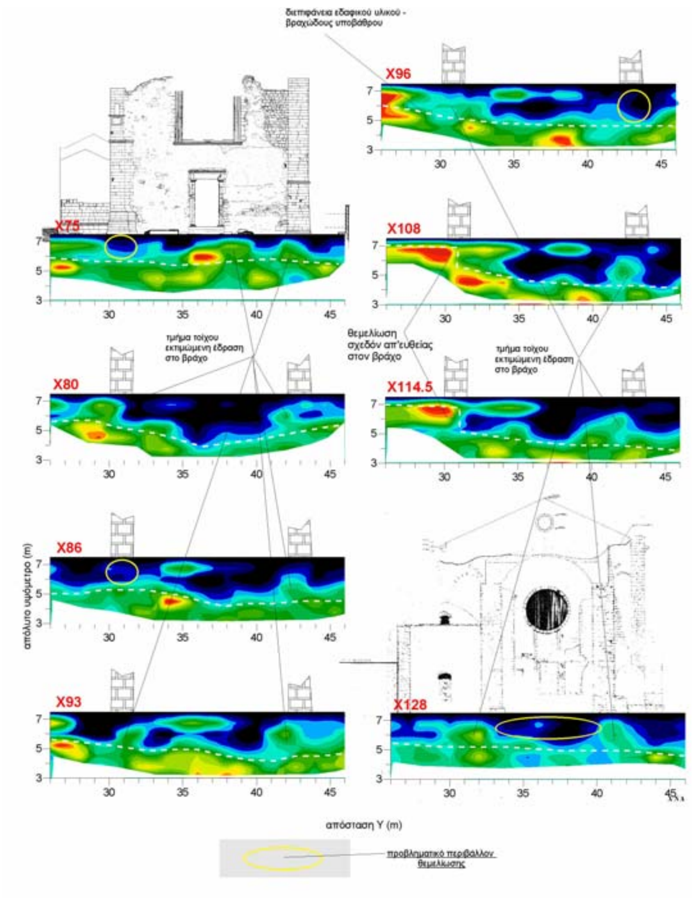


Figure 5. Vertical sections along the longitudinal axis of the Saint Peter of the Dominicans Church (Polymenakos & Tsatsanifos, 2003).

- Cutting of piles, in the case of deep foundations
- Jetting of the soil beneath the pile tips
- Jacking of the foundation on the "low" side
- Fracture grouting

ii. "Soft" methods, which rely on processes which produce corrective foundation movements by inducing appropriate ground movements, like:
- Soil extraction
- Dewatering
- Compensation grouting
- Removal of soil support

On the other hand, among the options that may be considered for foundation enhancement works are:

i. Repair of the existing foundations which contain imperfections or defects.
ii. Strengthening of the existing foundation by its extension or addition of new footings and shear beams connecting the footings in order to strengthen and / or stiffen the existing foundation and increase the foundation bearing area.
iii. Underpinning of the foundation by means of oscillated piles or by means of bored piles constructed through the body of the foundation.
iv. In case of pile foundations extension of the pile caps or rafts to provide additional bearing capacity and stiffness.
v. Increase of the footing level of the foundation.
vi. Subsoil improvement (cementation, silication, chemical and electro - chemical strengthening, high pressure grouting capable of stabilising the soil mass, deep soil mixing, etc.).
vii. Provision of a slab underneath the building or a box - type foundation in the underground area of the building.
viii. In order to protect existing buildings from the uneven settlements induced by excavations or construction of new buildings in their neighbourhood, isolation or separation trenches can be constructed between the two structures

The most famous example of the contribution of geotechnical engineering in the rehabilitation of a building is that of the Leaning Tower of Pisa, where the soil extraction method has been applied. The tower is founded on weak, highly compressible soils and its inclination has been increasing inexorably over the years to the point at which it was about to reach leaning instability (about 5.5 degrees to the vertical - see Fig. 6 from Burland et al., 2003).

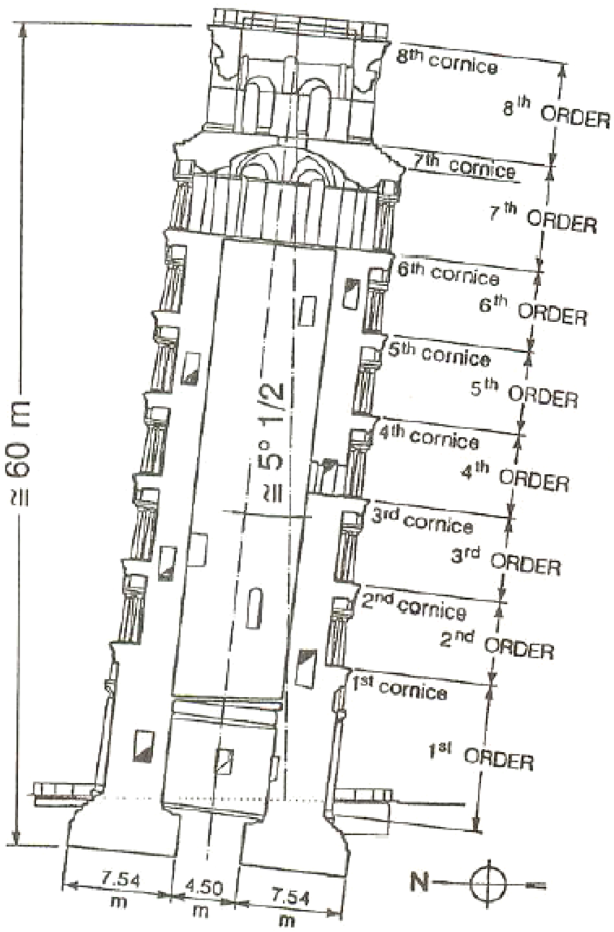


Figure 6. Cross section of the Leaning Tower of Pisa

Any disturbance to the ground beneath the south side of the foundation was very dangerous; therefore the use of conventional geotechnical approaches at the south side, such as underpinning, grouting etc., involved unacceptable risk. Since the internationally accepted conventions for the conservation and preservation of monuments and historic sites provided that any intrusive intervention on the Tower had to be kept to an absolute minimum, permanent stabilisation schemes involving propping or visible support were unacceptable and in any case could have triggered the collapse of the fragile masonry. After a careful consideration of a number of possible approaches, the International Committee for the Safeguard and Stabilisation of the Tower of Pisa, appointed by the Italian Government, adopted a controlled removal of small volumes of soil from beneath the north side of the tower foundation (underexcavation – see Figures 7 and 8). This technique provided an ultra soft method of increasing the stability of the tower, which is completely consistent with the requirement of architectural conservation.

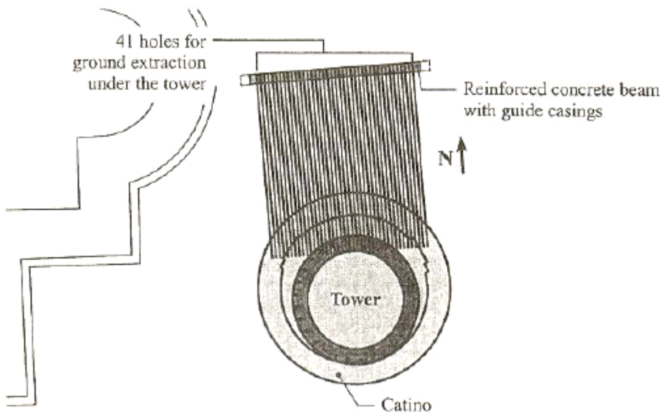


Figure 7. Pisa Tower. Holes for full ground extraction (Burland et al. 2003).

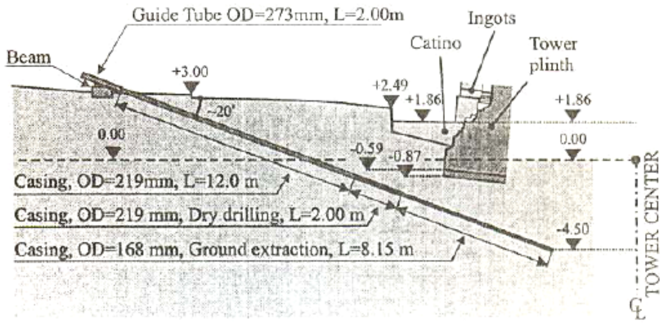


Figure 8. Pisa Tower. A hole for full ground extraction (Burland et al. 2003).

Different physical and numerical models have been employed to predict the effects of soil removal on the stability. The preliminary underexcavation intervention, only undertaken once the Commission was satisfied by comprehensive numerical and physical modelling together with a large scale trial, has demonstrated that the tower responds very positively to soil extraction. The final underexcavation has attained the target of reducing the tilt of the tower by half a degree, i.e. to bring the tower "back to future" to the time just before the excavation of the catino in 1838.

The technique of soil extraction has been used for rehabilitation of buildings longer before proposed by Terracina (1962) for Pisa. Johnston & Burland (2004) reported the application of the method as early as 1832 by James Trubshaw for the stabilization of the 15th century tower of St Chad's church in Wybunbury, South Cheshire. Barends (2002) gives a full contemporary account of the stabilization of a leaning church tower at Nijland by means of soil extraction in 1866. In the same year it has come to light that the method of soil extraction was also used to straighten a 100 m high chimney at the Bochum Cast Steel Works in Germany. The report on the work was discovered in the journal the '*Zeitschif Bauwesen*' published in 1867 and written by Haarman – the engineer who executed the work (see Figure 9, Johnston & Burland, 2004). Brandl (1989) has described the use of soil extraction to correct uneven settlement of piles supporting bridge piers, while the use of soil extraction has been widely used in Mexico City to reduce the differential settlement of a number of buildings due to regional subsidence and earthquake effects, before its application to the Pisa Tower (Tamez et al., 1997).

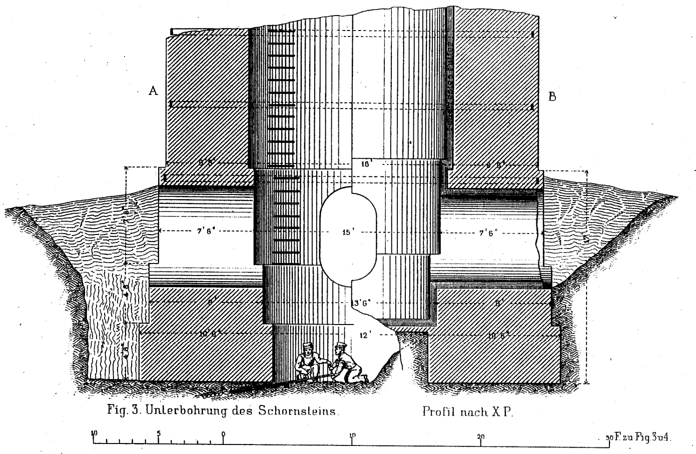


Figure 9. Vertical section at base of Bochum chimney showing the process of soil extraction (Johnston & Burland, 2004).

A similar to the soil extraction approach was proposed by Poulos et al. (2003) for the rehabilitation of buildings on piles which have undergone uneven settlements due to uneven ground conditions, or/and interaction among closely-spaced buildings, or/and faults in the foundation piling. The approach, which has been termed the "RSS" (Removal of Soil Support) method, involves the drilling of a number of boreholes on the "high side" of the building, so that restoring vertical movements will be developed within the area of the building foundation (see Fig. 10). A major advantage of the method is that it is not intrusive (i.e. it can be performed outside the building footprint) and can be controlled and adjusted via an observational approach.

A very interesting example of underpinning for strengthening the foundation of a historical building was presented by Sata (2003). The AEB Bank chose

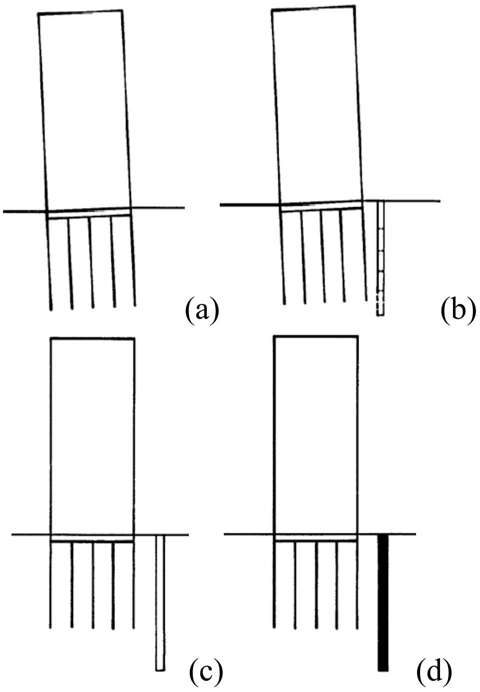


Figure 10. Principle of the RSS method: (a) Tilting of pile – supported structure (b) Progressive drilling of boreholes on the "high" side of the foundation (c) Restoration of structure tilt (d) Grouting of boreholes.

a two-storied historic building for its headquarters in Budapest (see Fig. 11). The renewal, re-utilisation and enlargement of the building should follow the original architecture. An underground garage had also to be constructed, requiring the deepening of the foundation level.

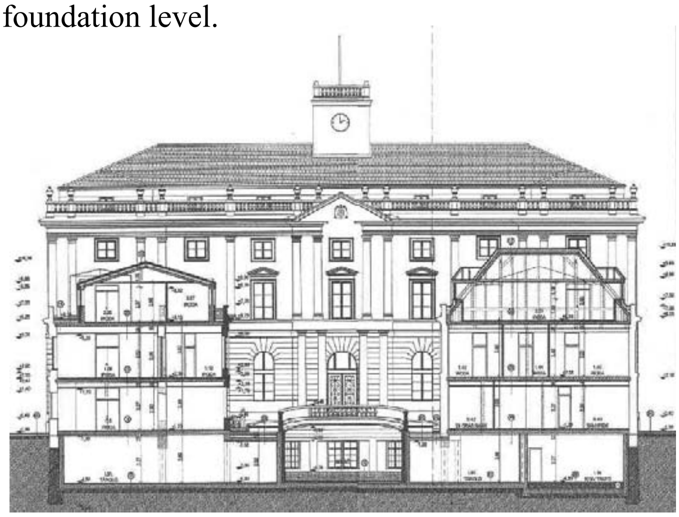


Figure 11. Architectural section of the renewed AEB Bank in Budapest.

Jet-grouting was used, and the whole intervention was executed as follows:

i. Reinforcement of the external walls, creating a deeper definitive foundation level – by using the jet-grouting technology and CFA piling.
ii. Creation of temporary supports for the main brick walls, by using the already mentioned jet grouting technology.
iii. Construction of the foundation of the final supports of the brick walls.
iv. Excavation and construction of the basement slab, construction of the final structure and removal of the temporary supports.

The loads of the internal walls were between 100 and 300 kN per meter and were transferred to the ground, temporarily, through micropiles (Figure 12).

Figure 12. The AEB Bank building "in the air"

In order to avoid any horizontal movements or / and vertical displacements of the very fragile brick-walls, jet piles were made on the two sides of the wall, and into them common steel tubes were placed. The connection between these so-called micro-pile heads and the wall is shown in Figure 13.

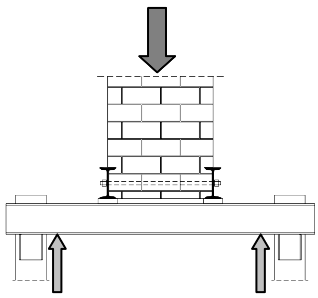


Figure 13. The connection between the micro-pile heads and the wall for the AEB Bank building.

Another very interesting case study of rehabilitation of the foundation of a building is that of the Sweden's Parliament, the Sveriges Riksdag.

The building has been constructed on the small island of Helgeandsholmen in the centre of Stockholm, over a century ago and is founded on timber piles. Regional uplift since the Riksdag was built means that the ground surface is higher, relative to sea level, than when its piles were installed. The top 700 mm of the timber piles were exposed to the atmosphere and were beginning to rot, with considerable risk of settlement as a consequence.

Two main solutions have been considered by the Swedish Parliament. Initial plans (a solution of the problem was sought since 1980) were to underpin building with new piles. More recently efforts have focused on a novel solution which involved creating a cut-off dam downstream aiming at restoring the groundwater to the level it was when the piles were installed, re-immersing the wooden piles in the groundwater and halting their degradation.

At the Riksdag the cut-off dam runs along the north side and then cuts across in front of its main façade, creating a curved L-shape (see Figure 14) that connects to the low weir, marking the transition between the lake and the tidal Baltic Sea.

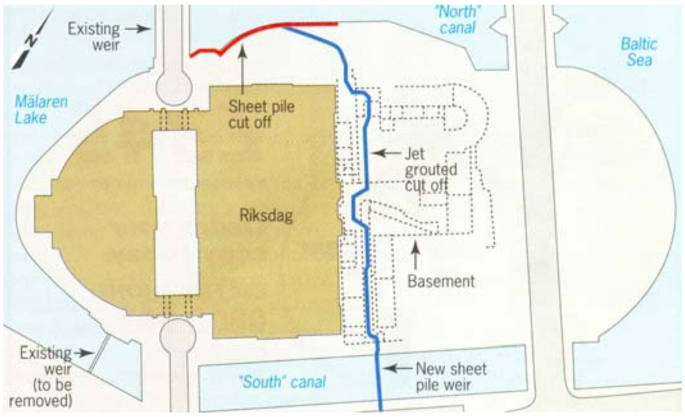


Figure 14. The cut-off dam to restore groundwater below Riksdag to the level of Lake Malaren

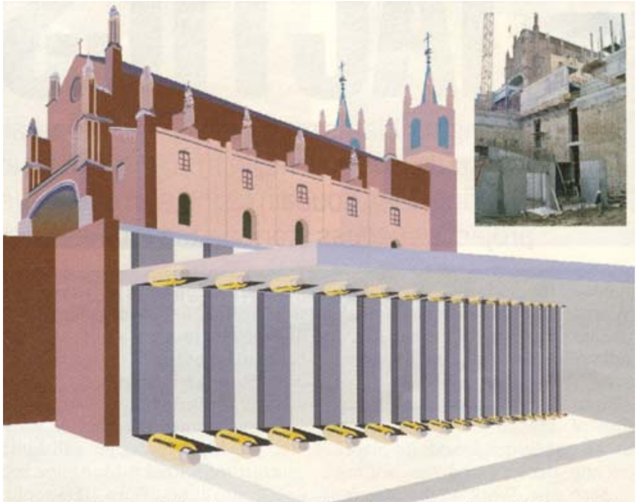
The dam is made up of two rows of jet grouted columns, extending from 15 m to 30 m in a sandy moraine and penetrating 150 mm into rock (European Foundations, 2004).

Madrid's Prado Museum is the largest art gallery in the world, but only has space to show a tenth of its immense collection at any one time. An expansion programme, completed in 2005, doubled the museum's capacity by constructing the spacious underground Jeronimos extension, which connects the museum's main neo-classical Villaneuva building to the existing cloister of the San Jeronimos church.

The most geotechnically demanding component of the project has been to create a 20 m deep, 46 m by 20 m hole beside these historic buildings and adjacent roads.

The ground movements were controlled with the novel use of 34 integrated hydraulic jacks that in effect reacted and pushed back the retaining walls as they started to move. The excavation was built top-down and jacks were installed to connect the two new "floating floors" to the retaining walls, incorporating a system which measured movement in the wall caused by the load of the surrounding buildings and traffic (see Figure 15).

At each of the two floor levels, separated vertically by 8 m, 17 double-acting 500 t capacity jacks were installed, each with 100 mm stroke. Individual



Integrated hydraulic jacks were placed in the top-down installed floor slabs.

Figure 15. The retaining system for the construction of the Prado Museum's underground extension

jacks were equipped with a positioning sensor to measure movement and a tilting swivel saddle to compensate for side load caused by non-centred loads (European Foundations, 2004).

As mentioned before, in rehabilitating old structures one should consider the existing foundations, and the impact of these old structures will have on modern geotechnical design. Existing foundations might perform satisfactorily under new loads, even though they are decades old. The research project "Re-use of Foundations on Urban Sites" (RuFUS), financed by the 5th European Commission Framework, has aimed at providing ways to overcome the barriers, both technical and non-technical, to the re-use of foundations for sustainable development. The outcome of the project was presented and discussed at the RuFUS Conference (19-20 October 2006) and published at the conference proceedings as well as in A Best Practice Handbook.

Figure 16. The RuFUS rosette.

Finally, rehabilitation of buildings in urban environments could create damages to adjacent structures. Following a spell of wet weather in February 2006, several residential properties in Burgess Hill, West Sussex, experienced foul flooding. Initially it was believed that the cause was a simple blocked sewer. However, detailed survey identified that part of the downstream sewer pipe, located 100 m from the area of foul flooding, had been damaged during the installation of pile foundations for a small house extension. The piles had breached the sewer pipe, located 7 m below ground surface, resulting in the 450 mm diameter sewer pipe being filled with 6 m of structural concrete.

The only solution with minimal disruption to the properties and at a low cost was an in situ repair of the damaged sewer. The solution involved sinking a 7.5 m deep mine shaft, from which a tunnel was driven. Steel sheeting was used for retaining the wall of the 3 m^2 shaft. The ground was made up of stiff clay to a depth of 5 m and mudstone to 8 m. Once the tunnelers reached the sewer pipe, the heading was driven in both directions, breaking out the damaged pipe. The erroneous pile foundation was then trimmed back and supported with a steel bridge that diverted its point load to below the invert of the sewer. After removing the blockage, the sewer was structurally realigned, increasing its lifespan by at least 50 years. The excavation was then backfilled with concrete (Ground Engineering, 2007).

4 REHABILITATION OF ROADS AND RAILWAYS

The continuous increase of traffic volume, as well as of the loads of the vehicles using roads and railways and the freight loadings themselves impose the need for their rehabilitation, mainly of the sub-grade – sub-base – pavement system.

Pavement rehabilitation problems may be enormously varied and range from the addition of rejuvenating surface treatment coats and simple overlays to total reconstruction. Rehabilitation due to normal traffic growth is usually solved by the use of overlays, whereas reconstruction work is necessary for pavements showing incipient failure (Rodriguez et al., 1988).

Establishing a criterion for rehabilitation is strictly a matter of reviewing the circumstances responsible for the unsatisfactory performance of the pavement. It is far more complicated than simply observing the appearance of superficial cracks. *Unsatisfactory* rarely is the result of a catastrophic failure. Rehabilitation may prove necessary in a pavement that is appropriately supporting very high volumes and loads of traffic, but for which maintenance costs are excessively high. The following are the principle criteria that are usually considered when determining the need for rehabilitation (Highway Research Board, 1972):

i. Level of serviceability, estimated, usually, on the basis of the opinion of a group of users.
ii. Structural condition, i.e. the capacity of the pavement to support current traffic loads and to continue to do so in the near future without progressive damage.
iii. Surface condition (irregularities, waves, ruts, and cracks, not necessarily associated with structural capacity).
iv. Safety usually assessed on the basis of accident statistics.
v. Cost, referring not only to the expenditure required for rehabilitation, but also to continuing maintenance and operational costs.

On top of these some less tangible factors, as the anticipated increase in traffic volume and vehicle loads to which the pavement will be subjected, the cost of rehabilitation work and the availability of funds for its execution, the service life anticipated or desired for the rehabilitation work and the cost of the pavement maintenance, must be considered.

The usual pavement reinforcement consists of an overlay of asphaltic concrete or a combination of asphalt concrete and layers of a granular material that can be stabilised or treated with asphalt, cement or lime. Geosynthetics can also be used.

5 REHABILITATION OF SLOPES

The Egnatia Odos project comprises the construction of a 680 km motorway from the port of Igoumenitsa, at the NW coast of Greece, through Macedonia and Thrace, to Alexandroupolis, a city situated near the border with Turkey. A big part of the motorway “cuts” through the mountains of Pindus and its “wild flysch” formations (melange type formations consisting of fractured siltstones with sandstone, limestone and clayey schist fragments, of variable size, in a siltstone matrix), resulted from the thrust of the Pindus geotectonic zone over the Ionian zone (Tsatsanifos & Pandis, 2005).

Near the village of Anthochori and the town of Metsovo the alignment of the motorway called for the construction of a 190 m long embankment. The geomorphology of the greater area as well as the “geological history” of that region of Greece, which is well known for frequent natural slope failures, were showing up the existence of old landslides, but were not taken into consideration when firstly designing the embankment. The first indications of the reactivation of an old landslide were revealed in the form of embankment settlement and culverts’ cracking, in an early phase of the project (1994). The works stopped at that time, due to change in the alignment of the highway, and restarted in 2000, when the temporary placement of excavation materials from a nearby cut reactivated again the preexisting landslide. Well defined cracks started showing on the ground surface “shortly” after the excavation material from the cut was placed. These cracks kept on propagating, quite rapidly, upslope at a point at which the boundaries of the whole landslide could be clearly identified.

Detailed geological and geotechnical investigations were carried out in order to define the geometry of the slip surface and the geomechanical properties of the soil layers above, below and along the slip surface itself. The geotechnical investigation program was carried out, in a step-by-step approach following evaluation of all data available as being acquired.

The sliding mass geomaterial comprised of "completely to highly weathered siltstone'' and "old landslide debris", in the form of low plasticity clay with sandstone – siltstone gravels, underlain by "moderately sheared" siltstone. The bedrock is in form of slightly sheared siltstone and fractured sandstone.

The inclinometer measurements revealed that the failure surface develops between the "highly weathered" and the "moderately sheared" siltstone at a depth ranging from 20 m to 28 m, approximately, with a rate of movement ranging between 7.9 x 10^{-5} mm/min and 6.3 x 10^{-4} mm/min (see Figure 17).

Back analyses yielded a factor of safety equal to FS = 1.114, well above 1.00, meaning that the placement of the excavations' materials was adding to the stability of the slope! The picture changed when excess pore water pressures were considered to have been developed within the low permeability clay formations underlying the fill area.

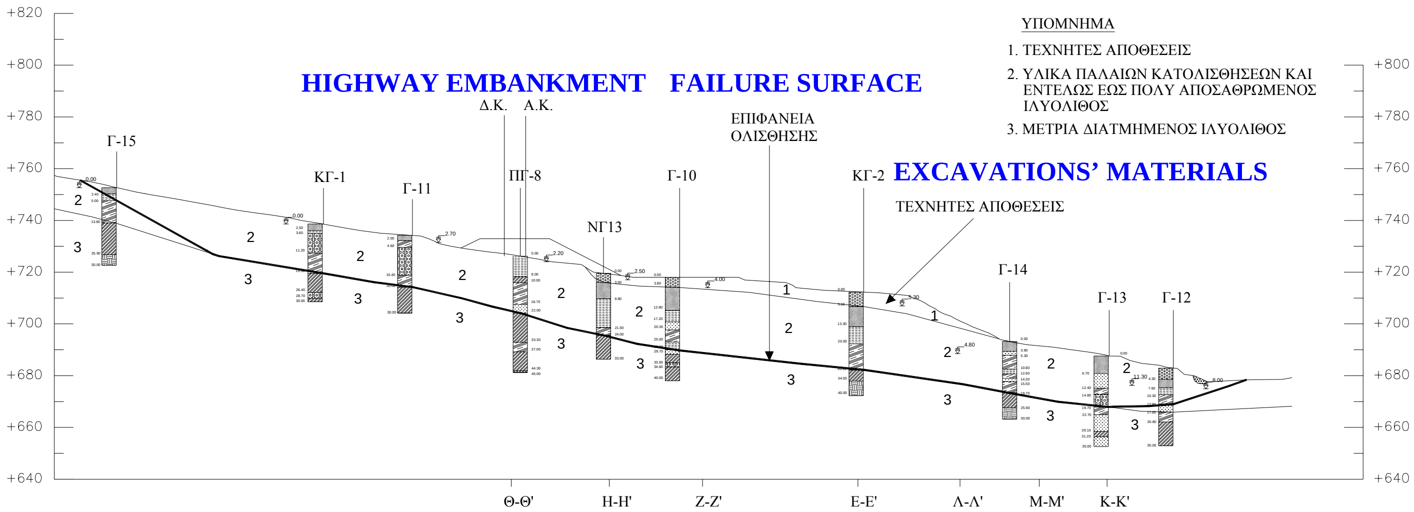


Figure 17. Slope section of the Anthochori Landslide with the assumed failure surface and the applied external loadings.

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It was found that the rapid ("undrained") placement of the excavation material and a raise in excess pore water pressures of the order of 70% (Ā = 0.70) of the total fill weight, along the part of the failure surface underlying the area where fill material had been placed, was sufficient to reinitiate the slide (FS = 0.993)(Tsatsanifos et al., 2006).

The stabilization measures selected comprised the construction of a berm at the toe of the landslide - a large retaining structure constructed by gabions along the bank of the Metsovitikos River - combined with a net of drainage ditches behind the embankment and a drainage blanket under the embankment. Also, on two ravine beds, passing through the area of the landslide, Armco tubes, able to accept differential settlements without being cracked, were placed under the stabilization fill. Finally, the slopes of the toe weight were vegetated for preventing their erosion. Figure 18 shows a general view of the landslide area after the construction of the embankment and of the remedial measures.

Figure 18. The Anthochori Embankment on the Landslide Area after the Construction of the Remedial Measures.

6 REHABILITATION OF RETAINING STRUCTURES

In 2003, a condition assessment of the retaining wall at Gibbon Falls, Yellowstone National Park, USA, established that it was losing its foundation and was in danger of failing. In addition to conducting a further investigation to better understand the condition of the wall and failure mechanism, it was decided to conduct a pilot micropile program. A micropile was drilled through the face rock of the stone masonry wall, which added strength and ductility to the wall, and also provided underpinning several meters into the wall foundation. The layer of "face rock" averaged 430 mm wide, with the narrowest measured point measuring approximately 300 mm. The mortar condition in the wall was unknown, yet anticipated to be highly variable. The contractor had to comply with several restrictions: no new grout could be visible, drill rods could not come through the outer wall face, the casing and bar could not be visible after completion, and the drill could not sit on, push against, or vibrate the wall during drilling.

The contractor drilled the micropile on the same batter as the wall face using high rotational speed (1,000 rpm), and low down pressure to help maintain the batter (Figure 19). Drilling progressed without problems through the 8.5 m wall, through 5 m of gravel and cobbles, and through 3 m of highly to moderately weathered rhyolitic tuff. Casing was installed and a 25 mm threadbar was grouted into the hole through perforations in the casing below the toe of the wall, under 3 MPa of pressure. When the work was completed, there were no visible signs to indicate that any significant disturbance had occurred. The pilea added reinforcement and bearing capacity and preserved the historic structure without impacting the natural resources around the wall (Barrows et al., 2005).

Figure 19. Micropile installation at Gibbon Falls Retaining Wall (Barrows et al., 2005).

7 REHABILITATION OF BRIDGES

The differential settlements, which were presented by the two central piers of a bridge over the Athens – Corinth Highway, is a classical example of required rehabilitation of a structure because of lack of geotechnical information combined with inadequate design practice (Stefanides & Tsatsanifos, 1995).

The design of the pile foundations of the bridge was based on the results of boreholes performed for a series of neighbouring structures, however at a distance of about 100 m (see Figure 20).

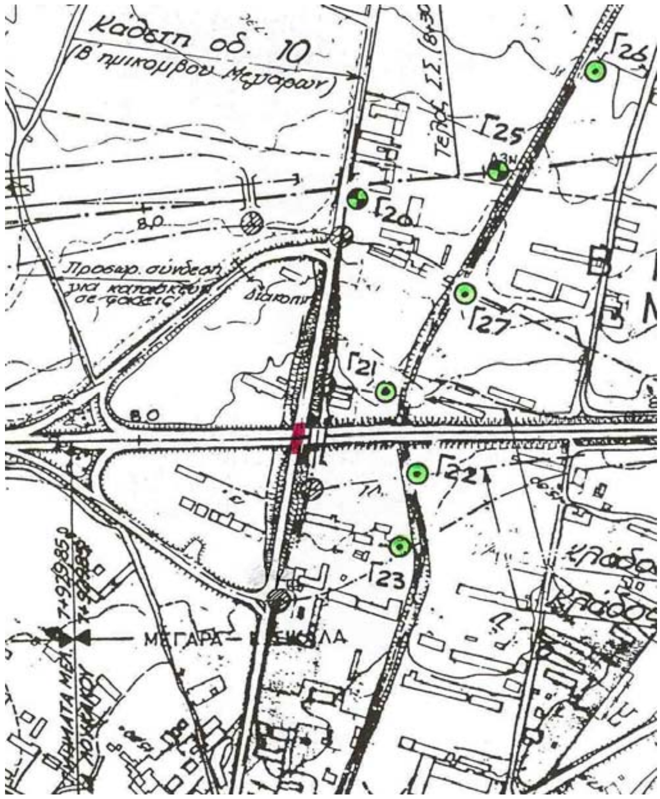


Figure 20. Location of bridge with differential pier settlements and of boreholes, on which the design was based.

The pile tips reached a depth of about 10.5 m below ground surface, according to the design, where a layer of dense gravels with sand has been foreseen. However, the design did not take into account that through that area a small stream was passing, having created, through its old meanders, different subsoil conditions within short distances. Thus, while the piles at the north pier reached the gravel layer, those of the south pier did not and after 15 months from the beginning of the measurements of the settlements the south pier had settled by 83 cm compared to the settlement of 16 cm of the north pier (see Figure 21).

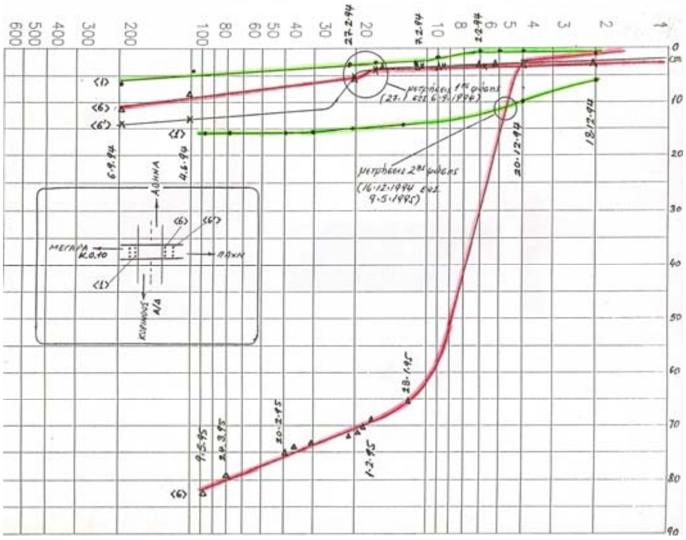


Figure 21. Settlements of the central piers of the bridge (red: south pier, green: north pier).

Five alternative methods were proposed for the rehabilitation of the bridge, taking into account that the motorway under the bridge should be in continuous use:

i. Use of small diameter piles and reconstruction of the pile cap.
ii. Construction of two series of small diameter piles at both sides of the pile cap and connection of them with the pile cap.
iii. Construction of diaphragm walls at both sides of the pile cap and connection of them with the pile cap.
iv. Extension of the pile cap and addition of new piles.
v. Use of jet grouting to improve the soil over and beneath the pile tips.

Finally, the decision was made to apply the last method and the bridge presents no settlements since then.

Nez Perce Creek Bridge in Yellowstone National Park USA (see Figure 22) was widened in 1996 to allow for increased traffic and larger vehicles, and to comply with current safety standards. It has been determined that the stone masonry piers would not sustain the design seismic loads but it was important to retain the look of the original work. One of the steps taken to accommodate these new design loads was to add steel reinforcement to the piers to make them more ductile and self-stable, and to isolate them from the bridge deck. This technique involved placing micropiles through each pier at 1.5 m spacing, and extending them 3 m into bedrock. The deck isolation was

Figure 22. Nez Perce Creek Bridge at the Yellowstone National Park, USA (Barrows et al., 2005).

achieved by elastomeric/steel bearing pads which allowed the pier masonry to remain untouched. The bridge abutments were widened by deconstruction of the stone masonry and constructing new concrete core walls for the additional width. To retain the same masonry look, the walls were faced with original stones as well as stones conserved from roadway excavation. (Barrows et al., 2005).

The Million Dollar Bridge carries the Copper River Highway across the Copper River near Cordova, Alaska, approximately 240 km east of Anchorage. The 478.5 m long bridge consists of four Pratt truss spans that, from south to north, measure 122 m, 92 m, 137 m and 122 m. The bridge is supported in the river on three immense concrete piers. The structure was built in 1909 and 1910. Its name comes from its construction cost, which exceeded \$1.4 million.

In 1964 the structure was severely damaged during Alaska's "Good Friday" earthquake (March 27, M = 9.2). During the quake the southern end of span 4 separated from pier 3 and fell into the river. Span 3 shifted several feet on top of pier 3; the pier itself tilted several degrees from the vertical, and its top part sheared several feet relative to the bottom part (see Figure 23). Because of this heavy damage, the authorities closed the structure to traffic.

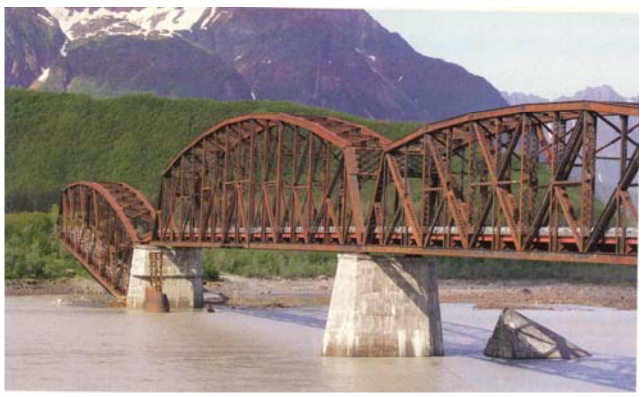


Figure 23. The Million Dollar Bridge after the Alaska's 1964 earthquake.

On March 31, 2000, the Million Dollar Bridge was added to the National Register of Historic Places, prompting officials to rehabilitate the bridge and perform a seismic retrofit to give it the ability to withstand a future earthquake.

The first major repair to the bridge involved raising the fallen span 4. This was accomplished by erecting temporary support towers upstream and downstream of the fallen span.

The other major facet of the bridge's rehabilitation involved replacing the damaged pier 3, which appears to have been constructed in a zone of liquefiable river sands. Liquefaction of the sands beneath pier 3 is believed to have been the main cause of the bridge's failure during the earthquake. Such liquefaction would explain the observed 5 degree tilt of the pier and the inferred tilt of the caisson supporting it. Indeed, span 4 may have fallen off the pier because of that tilt. As the span fell, it struck the pier and broke that structure along a construction joint.

To prevent a similar problem in future earthquakes, the solution seen as most cost effective was to replace the damaged pier with a hollow pier supported on piles that would extend 46 m below the riverbed, that is, below the zone of liquefiable sands (see Figure 24). The new foundation is supported on five piles, each a 1.8 m diameter steel pipe filled with concrete.

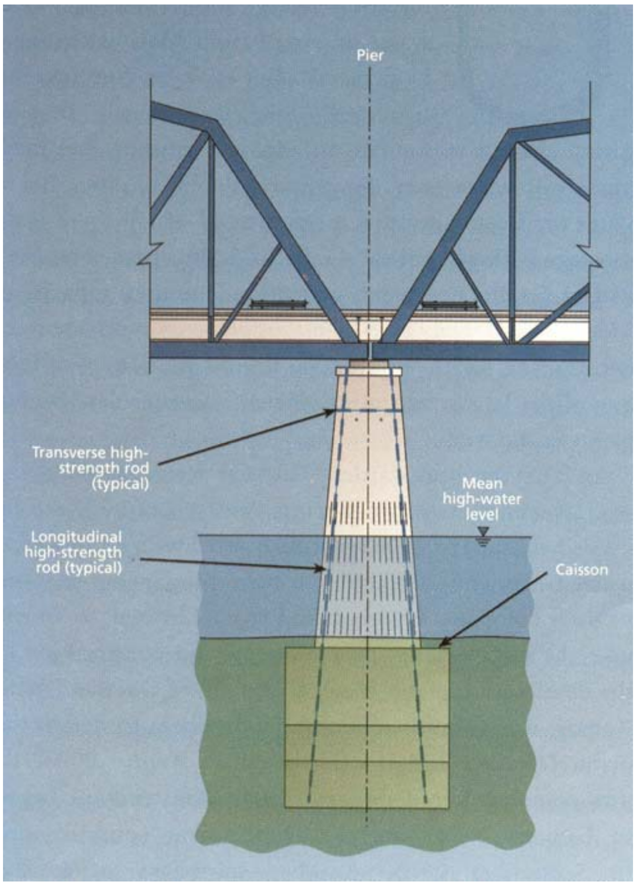


Figure 24. Pier 3 replacement at the Million Dollar Bridge, Alaska.

The seismic retrofit will involve installing friction pendulum isolation bearings, as well as strengthening piers 1 and 2 (massive structures of unreinforced concrete). At these piers, bundles of three high-strength steel rods – each rod 32 mm in diameter – will be grouted into 140 mm diameter holes cored through the piers and extending into the caissons below. The rods will increase the flexural strength of the piers and prevent rocking on the caissons that might damage the friction pendulum bearings (Ingham et al., 2007).

8 REHABILITATION OF TUNNELS

The 45 years old 619 m long George Massey Tunnel carries four lanes of traffic under the Fraser River to the south west of Vancouver, Canada. Built in an era when little consideration was given to seismic concerns, the tunnel is made of six 350 m long precast sections in a trench underlain by almost 600 m of loose saturated sediments.

Upgrading the tunnel is vital as it is designated lifeline structure on Vancouver's Disaster Response Route (a network of land and water transportation routes that will be open to emergency vehicles only in a post disaster situation).

Though the quality of the precast concrete was good, there was only about half the reinforcing that would be used today. This made the structure incapable of handling any relative movement produced by the new design earthquake.

Structural upgrade involved adding reinforcement to ensure that an earthquake would induce many well distributed cracks as opposed to a few large ones. The biggest challenge was finding space where to add steel. Trucks already "scrape" the ceiling so only the ceiling corners, the floors and the walls were reinforced, as well as the wind tubes and the air ducts (see Figure 25).

Figure 25. Crews prepare the main joint between tunnel sections for structural repair and strengthening of George Massey Tunnel.

Once the structural upgrades are complete the tunnel will be able to withstand the rigors of a separate contract to complete geotechnical upgrade. Earlier concerns of the tunnel floating during an earthquake resulted in rock and concrete being placed over the tunnel, which add to the challenge of underwater ground improvement. The plan is to improve by densifying a 50 ft wide strip along both sides of the tunnel (65 ft wide near the river bank) and to 30-45 ft below the mudline using stone columns or timber piles. Other measures to prevent movement include careful monitoring, and alternating the densification between both sides of the tunnel. (Jurbin, 2006).

The Tymfristos Tunnel has been constructed along the Lamia – Karpenissi (East – West) Road Axis of Central Greece. The 1,365 m long tunnel is passing through the flysch formation of the Pindos geotectonic unit, which consists, mainly, of claystones and slickensided argillaceous schists. It was designed to be excavated in two phases: top head and bench, without any provision for the distance between the top head and the bench excavation. Sprayed concrete (shotcrete), with wire mesh, steel frames and rockbolts were considered for the primary lining and a cast in place, steel reinforced, inner concrete lining.

Due to contractual problems and lack of funding the project has proceeded intermittently in three different contracts, with a halt of more than two years (from September 1995 to October 1997) between the second and third contract. During this period a 531 m long section of the top head, already excavated along the whole length of the tunnel, in an area with a rock mass of poor strength and deformation characteristics, suffered from excessive deformations, which destroyed the primary support and led to substantial closure of the opening (see Figure 26).

Figure 26. The maximum squeezing deformation of the Tymfristos Tunnel was as much as of 2 m.

In the process of rehabilitating the tunnel, a series of forth and back analyses were performed taking into account the new ground conditions. The analyses revealed that the deformation of the tunnel was due to the squeezing behaviour of the rock mass (Tsatsanifos et al., 2000) which could be tackled effectively through the “immediate” closure of the primary lining ring (Tsatsanifos, 1995). The excavation of the tunnel cross section along this part (130 m^2) was subdivided into top heading, bench and invert drifts, while three support classes were specified. Shotcrete (0.30 ÷ 0.40 m thick), with two layers of T131 wire mesh, lattice girders Pantex 70/30/D30 and systematic rockbolting (13/14 6 ÷ 9 m fully grouted bolts at each excavation step) were used for the primary lining. The excavation step of the top heading was 1 m and the complete closure of the primary lining ring was done every 2 m.

An extensive convergence - monitoring program has been adopted, providing for measuring sections every 15 m. Furthermore, a number of convergence stations have been installed in several sections ahead of the advancing face of the excavation, which greatly contributed to the adjustment of the support measures to the anticipated rock mass conditions.

9 REHABILITATION OF AIRPORTS

Airport operators face growing pressure to keep traffic moving on the ground while maintaining runways, taxiways and aprons. An added complexity is the need to assess and upgrade pavements for increased loadings from bigger planes. In this case one can apply the same methods already described for the rehabilitation of highway pavements.

Figure 27 shows a location of the geotechnical investigations performed for the design of the rehabilitation of the Tanagra Airport Runway Pavement. The results of the investigations and the subsequent preliminary design revealed that it was more economic to reconstruct the whole pavement

Figure 27. Geotechnical investigations for the rehabilitation of Tanagra airport runway pavement.

instead of rehabilitating it (Magrioti et al., 2003). The tricky think in this case was that, since Tanagra airport is a military airport in a 24 hours operation, the time allocated for the investigations was only one day, hence all the boreholes and trial pits had to be drilled / excavated and the pavement reinstated within this one day. To tackle this problem, special materials for the local rehabilitation of the pavement were used.

10 REHABILITATION OF PORTS

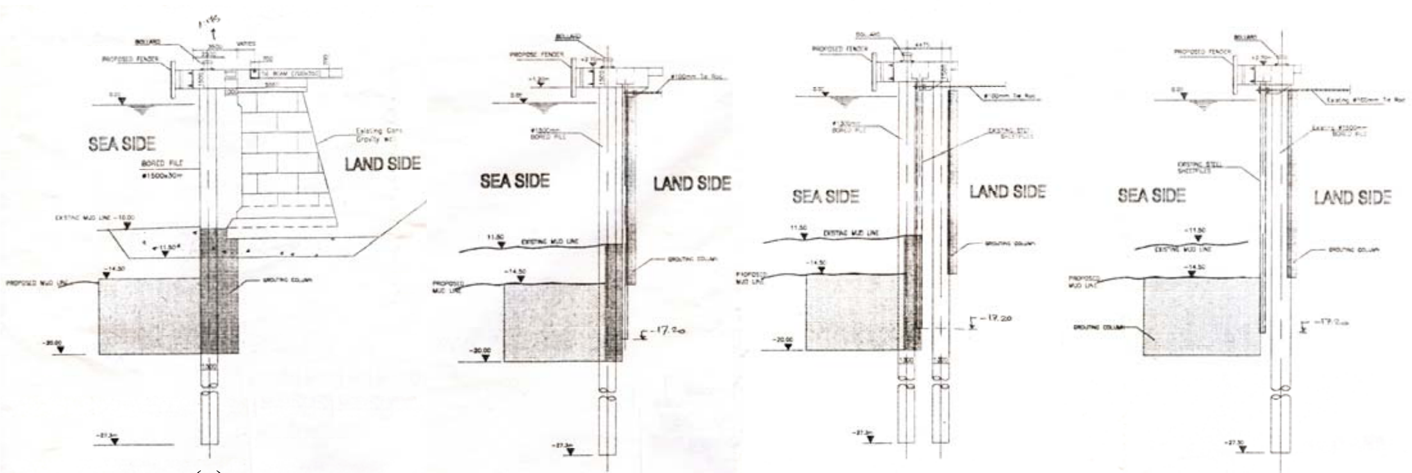
The Flour Mills of Nigeria, Inc. port installations at the Apapa Bay, Lagos, Nigeria consist of a pier of 435 m length, hosting docks and several storage houses and other installations necessary for cargo handling. Docking bollards are developed along the whole length of a 210 m long gravity wall, built with reinforced concrete, and a 225 m long Larssen sheet-pile wall. The average sea depth, in front of the dock, is of the order of 9.80 m, thus limiting the ship classes the port can accommodate to those with relative small sinking depth.

The rehabilitation of the whole port installations was found to be necessary on the basis of the increasing future needs. In that context, the installations should get modified in a way that the port will become capable of docking ships of 75,000 DWT in size. In order to do so, it was imperative to increase sea depth in front of the dock from 9.80 m to 14.50 m. However, given the geotechnical conditions prevailing in the area (soft silty – clayey deposits), such an increase of the sea bottom depth without taking special retaining measures could result in severe instability problems of the existing piers, because of the loss of the passive resistance that the soil exerts on the “foot” and the embedded length of the gravity walls and the sheet piles, respectively. There was a possibility of a deep circular failure developing underneath the foundation of the gravity wall, whereas in the case of the sheet pile walls such an action could result in excessive deformations at the top of the wall and / or at its embedded part.

In order to make possible the excavation of the sea bottom, a row of bored piles, to be placed prior to the excavation, was proposed, ensuring the stability of the existing walls. These piles were constructed in front of the gravity wall, whereas in the case of the sheet pile wall the initial proposal was to place bored piles on both sides of the wall, in order to achieve alignment of the pier face. However, unsurpassed difficulties were encountered behind the sheet pile wall during the construction of the bored piles. So, in order to overcome these difficulties, an alternative was proposed consisting mainly of stabilizing the soil, behind the existing diaphragm, by “soil-grout” piles, using the Jet Grouting method.

The single bored piles wall consist of piles of 1.50 m diameter, placed at 2.52 m, distance (from centre to centre), whereas along the transitional section of the pier, where two rows of bored piles wall were constructed, these have a diameter of 1.30 m, placed at 2.52 m, distance (from centre to centre). All piles reached the depth of -27.30.

In addition, a special arrangement for the pile cap was foreseen, in order to accomplish cooperation of the existing pier wall with the additional retaining measures, as well as anchoring, so that a part of the horizontal loading (caused by the waves, the wind etc.) is transferred to the ground.



(a) (b) (c) (d)

Figure 28. Different sections along the Apapa Bay, Lagos, Nigeria quay wall: (a) Gravity wall area, (b) Steel sheet-pile A area, (c) Steel sheet-pile B area, (d) Steel sheet-pile C area

According to the alternative proposal, two rows of “soil-grout” piles were constructed, behind the sheet pile wall, in contact with each other from level +1.20 (bottom side of the pile cap) down to level -22.00 (coincides with the foundation level of the external “box” of “soil-grout” piles) and a pile cap, on top of which the bollards were placed. Existing anchors were connected to the "new" pile cap as suggested in the initial proposal. Bollard load is primarily borne by the existing anchors and secondarily by shearing between the pile cap and the subsequent two rows of "soil grout" piles, which were ensured through placement of HE steel sections or steel tubes.

Behind the aforementioned two rows of piles, another group of "soil-grout" piles were placed, forming a square pattern cell, of 4.80 x 4.80 m^2, from level -2.00 down to -20.00.

Finally, a row of "struts" was constructed in 4.80 m distances, consisting of three tangential bored piles each, from -10.00 down to -20.00.

A jet grouting scheme was also foreseen in order to improve soil properties in front of the pier down to -20.00 m, so that the minimum accepted value of 1.50, for the factor of safety against general failure of the "soil-pier" system, is achieved

11 REHABILITATION OF WATERWAYS

The Corinth Canal is a junction of international sea transport and serves ships coming from the Western Mediterranean and Adriatic en route to Eastern Mediterranean and Black Sea ports and vice versa (see Figure 28). The Corinth Canal intersects the Isthmus of Corinth and has a length of 6.343 m. The minimum width of the canal at sea level is 24.6 m and bottom width of 21 m at 8 m depth.

The ancient seafarers, in order to avoid the circumnavigation of Peloponnesus, had to transport their entire ships and precious cargo intact across land from shore to shore, sliding them on a masonry trail known by the name of Diolkos.

Since early times, a number of spirited souls entertained thoughts of constructing a canal through the Isthmus. In 602 B.C., Periander, Tyrant of Corinth and one of the Seven Sages of Antiquity, was the first man to seriously consider the possibility of opening a canal through the Isthmus. Periander is said to have given up on his plans fearing the wrath of the gods. In 307 B.C., Demetrios Poliorketes made up his mind to cut a naval passage through the Isthmus. He actually began excavations before he was talked out of continuing with it by Egyptian engineers, who predicted that the different sea levels between the Corinthian and the Saronic Gulfs would inundate Aegina and nearby islands with the sea. In Roman times Julius Caesar in 44 B.C. and Caligula, in 37 B.C. again courted with the idea. In 66 A.D., Nero reconsidered earlier plans and, a year later, he set teams of war prisoners from the Aegean islands and six thousand slave Jews to work on the canal. They dug out a ditch 3,300 m in length and 40 m wide, before Nero had to rush back to Rome to quell the Galva mutiny. The next historic personality to be associated with the canal of Corinth was Herod of Atticus. He tried, as also did the Byzantines, but to no avail. The Venetians were next in line. They commenced digging from the shore on the Corinthian Gulf but the enormity of the task made them give up overnight.

Figure 28. The Corinth Canal and its location.

In the nineteenth century, Capodistrias, the first Governor of the Hellenic State, commissioned a special study on the canal project. The conclusions of that study made Capodistrias abandon further consideration. However, a final push of sufficient threshold energy came to rescue: Another mammoth-scale canal project, the Suez Canal opened its gates to naval traffic in 1869. In view of that event, in November 1869 the Hellenic Administration assigned the project to two French contractors, who, however, never started it. Twelve years later, in 1881, another contractor, a Hungarian general by the name of Stefan Tyrr took over the project. Construction of the canal began on April 23, 1882 and was completed in 1893. By then, the initial contractor had run dry of funds and was replaced by a Greek Company under Andreas Singros. Naval traffic in the Corinth Canal was inaugurated in a brilliant ceremony held on July 25th, 1893. It was indeed a vindication of a dream first conceived some 2495 years ago.

The rock formations in the flanks of the Corinth Canal consist of Neocene deposits of marls and sandstones. There are several faults running in east -west direction at a perpendicular angle to the canal axis, some of them seismically active. These geologic features were responsible for a number of major landslides into the Canal at several instances. On account of these landfalls, the Canal often had to be closed for repairs. The most serious such incident happened in 1923, when the Canal remained closed to traffic for 2 years on account of 41,000 m^3 of earth which had fallen in. Another major interruption of operation occurred in 1944, when the retreating German Army set explosives to the flanks of the Canal and caused 60,000 m^3 of earth to cave in. To make repairs even more difficult, the Germans also sunk railroad cars into the canal. It took 5 years to clear the Canal for traffic then.

In all these cases major rehabilitation works had been required for the re-opening of the Canal. Currently, in order to safeguard the passage of the ships through the Canal, continuous rehabilitation of the slopes is required, as shown in Figure 29.

Figure 29. Rehabilitation works on the slopes of the Corinth Canal

12 REHABILITATION OF DAMS

Public safety demands that earth dams either retain their reservoirs in the event of an earthquake or at worst release their reservoirs in a manner that does not pose a threat to life. Existing dams, especially older ones, are now being re-examined using more severe earthquakes that their designers envisioned with evaluation methods unavailable to the original designers. Some of these re-examinations result in the determination that a seismic hazard exists (Marcusson III et al., 1996).

In order to rehabilitate an earth dam to prevent potential seismic instability, one must either change the engineering properties of the dam and / or foundation, modify the geometry of the existing dam, or both.

Upstream and downstream berms and buttresses are used to increase the effective overburden pressure on the problem material and thus decrease its liquefaction potential, or to increase the failure surface, provide a counterweight to limit movement and maintain a remnant section.

In some cases excavation and replacement of the problem material is used, as well as in-situ densification or strengthening. Both cases result in the decrease of the liquefaction potential of the problem material involved. Similarly, drainage to relieve of seismically induced pore-water pressures decreases the liquefaction potential.

Finally, an increase in freeboard may, also, be used for rehabilitating old dams or various combinations of the aforementioned approaches. Nevertheless, there are cases where removal of the dam from service or replacement with a new one was the only feasible solution.

The Saluda Dam is a hydroelectric dam located approximately 10 miles west of Columbia, South Carolina, USA. It was built during the late 1920s (completed in 1930) on the Saluda River. It has been determined that a recurrence of the 1886 Charleston Earthquake (magnitude 7.3-7.5) would cause the Saluda dam to liquefy and fail. Such an earthquake is expected once every 450 years. The Lake Murray reservoir, behind the Saluda Dam, contains 2,200,000 acre-feet of water. Failure of the Saluda Dam would cause major flooding for many miles downstream, including the city of Columbia, flooding a population of over 100,000, with an expected significant loss of life.

The Saluda Dam Remediation Project required the construction of a new back-up dam, or an additional berm at the downstream toe of the dam to add stability. The new berm involved a 4 million cubic yard rock berm section and a 1.3 million cubic yard roller-compacted concrete section (Figures 29 and 30).

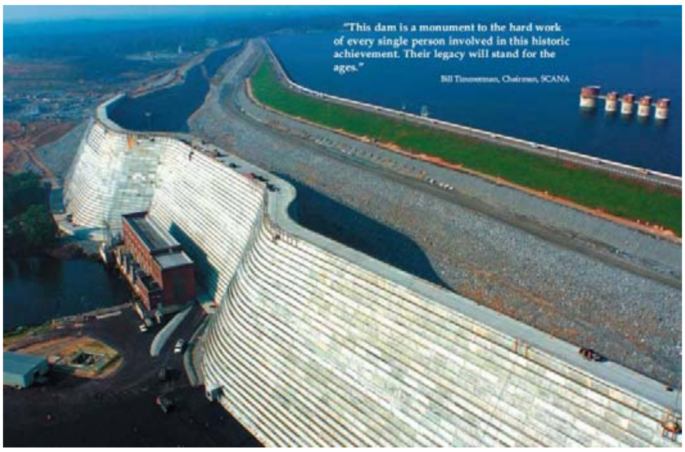


Figure 29. The Saluda Dam after its rehabilitation.

During the inspection in the process of the rehabilitation of the City of Baltimore's Loch Raven Dam, Maryland, USA, one of the main safety issues concerned was the need to inspect conditions

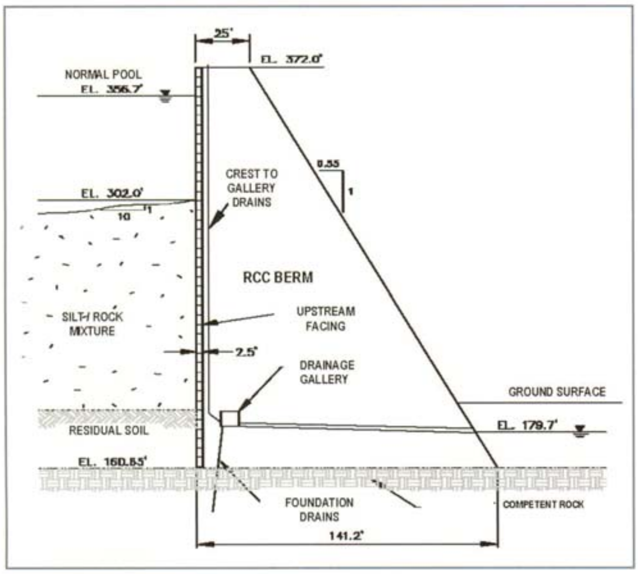


Figure 30. Saluda Dam's typical RCC berm section (Bair and Koleber, 2006).

of the dam's downstream toe to determine if excessive scouring and undermining of the structure had occurred.

To allow construction of the required stilling basin, significant excavation was required at the toe of the original (1912) structure. As this excavation would have decreased the dam's overall stability during construction, 57 high-capacity post tensioned rock anchors were designed to pin the dam to the underlying bedrock. Installed to resist potential loads created during construction, the anchors also act in combination with the RCC buttress section to stabilize the dam against the extreme loading condition that would occur in the event of a Probable Maximum Flood.

The anchors were installed within the spillway and a portion of the dam's nonoverflow sections. Typically they were installed through the dam's face and angled down at 45 degrees to penetrate the bedrock. Drilled holes were 300 to 350 mm in diameter and up to 49 m long. Total anchor lengths, including boded and unbonded sections, vary between 21 m and 48 m, the longest anchors being situated within the spillway (Bingham & Holderbaum, 2007).

The Forest Lake Reservoir supplies recycled irrigation water to seven Californian golf courses. The reservoir was originally constructed in 1887, and had operated as an unlined reservoir until the early 1990s. The rehabilitation of the project was commissioned by the authorities, because if water saturated the reservoir embankments, the stability of the embankments could be affected in an earthquake. The rehabilitation of the reservoir consisted in the application of a Hypalon liner (DiAntonio, 2007).

Standley Lake Dam and Reservoir in the northwestern greater Denver Metropolitan area, Colorado, USA, is an earth embankment with a height of 33.5 m and a crest length of 2,012 m. The reservoir stores 51.8 Mm^3 of raw water that cannot be drained without causing a major interruption to Denver's water service. The dam was constructed in 1908 using soil from railroad trestles and "puddling" in a clay core on a foundation of weak expansive, and slickensided claystone bedrock (Coss, 2006).

Given these factors, the dam was plagued with slope failures from the beginning. Recently, the old pressurized outlet works constructed through the maximum section of the dam, experienced separation problems at the joints due to creep and sliding of the embankment and foundation.

The rehabilitation of the dam included constructing outlet works tunneled, using microtunnelling techniques, in the abutment (separate from the embankment), a new spillway, and placement of additional stability berms (see Figure 31).

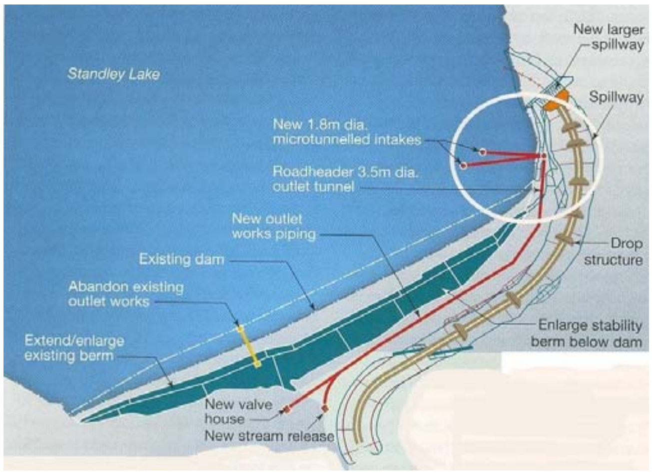


Figure 31. The Standley Dam Rehabilitation project (Coss, 2006).

Finally, sometimes it is not worth to rehabilitate an old dam, as in the case of the Calaveras Dam near San Francisco, USA. In 2001 the safety inspectors concluded that the dam (a hydraulic fill dam, with slopes buttressed with rockfill, completed in 1925) would probably collapse during an earthquake. The engineers considered a variety of alternatives, from rehabilitating the old dam to constructing a new one with the aid of such materials as roller-compacted concrete and earth and rock fill. They finally decided that the best and most economical solution would be to build a new earth dam just downstream of the existing dam (see

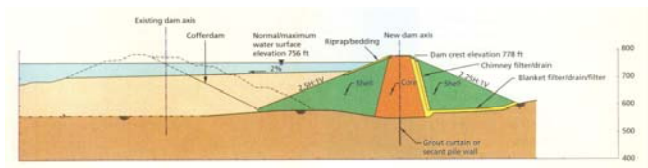


Figure 32. The Calaveras Old and New Dam, San Francisco (Hansen, 2005).

Figure 32) that will be able to withstand forces from a large earthquake from the nearby Calaveras Fault (Hansen, 2005).

13 LAND REHABILITATION

Land rehabilitation is the process of returning the land in a given area to some degree of its former self, after some process (business, industry, natural disaster etc.) has damaged it. Many projects and developments will result in the land becoming degraded, for example industry, mining, farming and forestry. Reusing brownfield sites is also a key to building the homes and commercial developments we need for the future.

While it is rarely possible to restore the land to its original condition, the rehabilitation process usually attempts to bring some degree of restoration. Modern methods have in many cases not only restored degraded land but actually improved it, depending on what criteria are used to measure "improvement".

Mine rehabilitation aims to minimize and mitigate the environmental effects of modern mining, which may in the case of open pit mining involve movement of significant volumes of rock. Rehabilitation management is an ongoing process, often resulting in open pit mines being backfilled.

After mining finishes, the mine area must undergo rehabilitation:

- Waste dumps are contoured to flatten them out, to further stabilise them against erosion.
- If the ore contains sulphides, it is usually covered with a layer of clay to prevent access of rain and oxygen from the air, which can oxidise the sulphides to produce sulphuric acid.
- If the ore contains asbestos, it is also covered with a layer of clay to avoid spread of asbestos fibres in the air.
- Waste dumps are covered with topsoil, and vegetation is planted to help consolidate the material.
- Dumps are usually fenced off to prevent livestock denuding them of vegetation.
- The open pit is then surrounded with a fence, to prevent access, and it generally eventually fills up with ground water.
- Tailings dams are left to evaporate, then covered with waste rock, clay if need be, and soil, which is planted to stabilise it.

For underground mines, rehabilitation is not always a significant problem or cost. This is because of the higher grade of the ore and lower volumes of waste rock and tailings. In some situations, stops are backfilled with concrete slurry using waste, so that minimal waste is left at surface.

The removal of plant and infrastructure is not always part of a rehabilitation programme, as many old mine plants have cultural heritage and cultural value. Often in gold mines, rehabilitation is performed by scavenger operations, which treat the soil within the plant area for spilled gold using modified placer mining gravity collection plants.

Another form of land rehabilitation is the restoration of ecosystems and landscapes, which have been altered because of infrastructure works. They are designed not only for the geotechnical stabilization but mainly to integrate the infrastructure in the local ecological context. Ecological restoration at the sides of a highway, for example, would lead to biodiversity, landscape and natural heritage conservation.

The rowing events of the Athens 2004 Olympic Games were held at the Olympic Rowing and Canoe Centre at Shinias, Marathon, in an area that used to be an airfield for light aircrafts and a military base for the last 40 years. However, before the construction of the airfield, the area used to be a marshland, very rich in wild life, which was drained in the late 1920s to create cultivatable land.

The design of the Olympic Centre (see Figure 33) presented some very interesting geotechnical problems to deal with and provided all the necessary works in order to reinstate the old marshland through the water overflow from the rowing ponds to the marshland (Tsatsanifos, 2004).

Figure 33. The Athens 2004 Olympic Games Rowing and Canoe Centre at Shinias, Marathon.

A similar story has been reported for a wetland in California. Before 1932, when it became the site of a military base, the land now occupied by the Hamilton Army Airfield, on the shore of San Pablo Bay near Novato, California, was strictly agricultural. But before it was reclaimed for agriculture at the beginning of the 20th century, the 1,000 acre parcel of land supported an array of tidal and seasonal wetlands. Now, more than 100 years later, the site is to be returned to its original state. The airfield's runway will remain in place, but its presence will not be noted as it will repose beneath several million cubic meters of dredged material that will be placed there (Landers, 2005).

14 CONCLUSIONS

An attempt to show the contribution of geotechnical engineering in the rehabilitation of buildings and infrastructures is made in this paper through the presentation of recent relative case histories. It is easily foreseen that rehabilitation initiatives are a key growth area for geotechnical innovation.

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